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THE PURPOSE OF THIS STUDY WAS TO CONSTRUCT, CALIBRATE, AND VERIFY A DIGITAL GROUND WATER MODEL FOR USE AS AN OPERATIONAL MANAGEMENT AND DESIGN TOOL FOR THE NORTH BOUNDARY CONTAINMENT TREATMENT SYSTEM.

CALIBRATION CONSISTED OF DEVELOPMENT OF THE PREDICTIVE CAPABILITIES OF THE MODEL BY INTERACTIVELY REFINING THE MODEL PARAMETERS TO CLOSELY APPROXIMATE OBSERVED PRE-BARRIER GROUND WATER CONDITIONS. THE MODEL WAS FURTHER REFINED BY REPLICATION OF HISTORICAL BARRIER SYSTEM OPERATION AND GROUND WATER CONDITIONS. THE CALIBRATED MODEL WAS UTILIZED TO SIMULATE OPERATION OF THE CURRENT SYSTEM, AS WELL AS TO PREDICT RESULTS OF SEVERAL OPTIONS FOR SYSTEM REDESIGN.

A THREE DIMENSIONAL CONTAMINANT TRANSPORT VERSION OF THE MODEL WAS USED TO INVESTIGATE POTENTIAL MIGRATORY PATHWAYS BETWEEN THE ALLUVIAL AQUIFER AND THE DENVER FORMATION. THE RESULTS SUGGEST THAT THE CONTAMINATION OBSERVED IN THE DENVER FORMATION IS BECAUSE OF LOCAL VERTICAL MIGRATION FROM THE ALLUVIAL AQUIFER.

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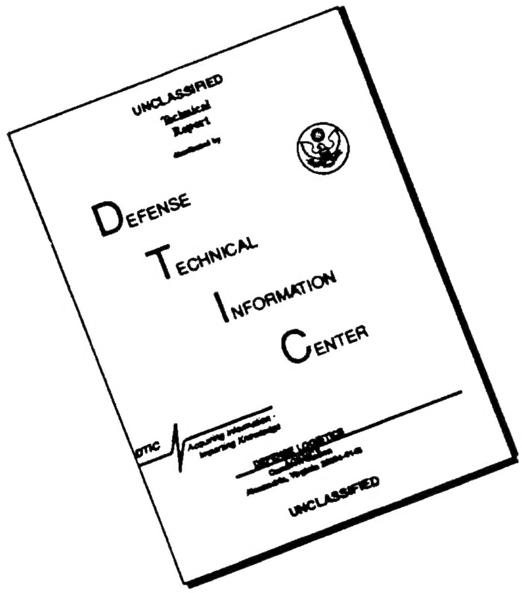

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DIGITAL OPERATIONAL MANAGEMENT MODEL OF THE NORTH BOUNDARY BARRIER SYSTEM AT THE ROCKY MOUNTAIN ARSENAL NEAR DENVER, COLORADO

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Preface

This report is one of a series of groundwater reports published by the Groundwater Program, Department of Civil Engineering, Colorado State University. CSU is one of the leading institutions in the nation in the field of groundwater education and research. The Groundwater Program at CSU has over a 30-year history. Areas of expertise include groundwater contamination from hazardous wastes, aquifer restoration, immiscible fluid flow (hydrocarbon contamination), artificial recharge, conjunctive use of ground and surface waters, numerical groundwater modeling, as well as expertise in many other areas of concern in groundwater. This groundwater report series was started in 1986 in order to unify the many reports that are published by the Groundwater Program at CSU on the various studies conducted at the University into groundwater problems of state and national concern.

This report describes the use of the Colorado State University finite element groundwater flow model CSU-GWFLOW as an operational management tool to manage the north boundary barrier system at the Rocky Mountain Arsenal. The mathematical development of CSU-GWFLOW is given in Groundwater Technical Report #2. Any questions concerning this study should be directed to the Groundwater Program, Department of Civil Engineering at Colorado State University.

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ABSTRACT

For nearly four decades, the Rocky Mountain Arsenal was utilized for the production, assembly, and finally, destruction of toxic and incendiary munitions. Parts of the site were leased to private companies for the manufacture of chemical pesticides. Disposal of chemical by-products and accidental releases of chemicals resulted in contamination of the surficial alluvial aquifer in and around the arsenal. In order to halt the spread of contaminated groundwater, the Army and Shell Chemical Company installed three groundwater barrier systems to block critical contaminant migration pathways until source controls could be implemented. The first of the groundwater barrier systems was installed at the north boundary of the arsenal in 1978.

The north boundary containment system consists of arrays of discharge and recharge wells separated by a bentonite slurry wall. The Army desired to examine management plans that would improve system operational flexibility and overall performance of the system. A primary goal of system operation is to minimize, or reverse the hydraulic head differential across the slurry wall.

The study consisted of construction, calibration, verification, and application of a digital finite element groundwater model (CSU-GWFLOW) to aid in answering the questions posed by the Army. Calibration consisted of development of the predictive capabilities of the model by interactively refining the model parameters to closely approximate observed pre-barrier groundwater conditions. The predictive ability of the model was further refined and defined by replication of historical barrier system operation and groundwater conditions. Approximately 79 monitoring wells were

represented in the model by nodal points in the finite element mesh. Documentation of the predictive capabilities of the model are provided in 10 tables which summarize comparison of observed and model predicted potentiometric levels for the observation points. Statistical summaries of model performance are provided.

The calibrated model was utilized to simulate operation of the current system as well as to predict results of several options for system re-design. Results of these simulations include pumping schedule for operation of the present and reconfigured system at the natural groundwater flow rate currently being intercepted by the system. Present and hypothetical pumping strategies and system configurations are compared and contrasted. The primary means of comparison is the hydraulic head differential across the slurry wall estimated to result from implementation of a given strategy. Both magnitude and distribution of the estimated head differential across the slurry wall are provided to assist the Army in making decisions concerning the future of the north boundary containment treatment system.

A three-dimensional contaminant transport version of the model was applied to the North barrier system in the vicinity of the original pilot scale system. This three-dimensional model included both the alluvial aquifer and the upper part of the Denver Formation. It was used to investigate potential migratory pathways between the surfaced alluvial aquifer and the underlying Denver Formation. This modeling effort indicated that lateral migration of contaminants in the Denver Formation is very slow. The model results suggest that the contamination observed in the Denver Formation is the likely result of local vertical migration from the overlying alluvial aquifer. Vertical migration times on the average were much less than lateral migration times.

The major focus of this study was the two-dimensional modeling of the groundwater, in great detail, flow system in the vicinity of the North boundary barrier system at the Arsenal. Contaminant transport was not modeled in this part of the study. In developing management strategies for the North barrier system knowledge of contaminant concentrations intercepted by barrier system is needed. This includes a knowledge of the concentration and distribution of contaminant plumes intercepted by the barrier system and how different barrier management scenarios would affect the flow paths, rate of travel and concentration of these plumes. Future modeling of contaminant transport at the North Boundary barrier should be considered for obtaining this valuable insight.

CHAPTER 1

INTRODUCTION

1.1 Background and Purpose

The Rocky Mountain Arsenal (RMA) consists of approximately 27 square-miles of land, about nine miles northeast of central downtown Denver, Colorado (Figure 1.1). The property has been largely controlled by the U.S. government since it was purchased in the early 1940's. The site was utilized for the production, storage, and finally, destruction of chemical and incendiary munitions. Part of the property was leased to private concerns for the manufacture of various chemical compounds including pesticides. Contamination of the alluvial aquifer occurred as a result of waste disposal practices and accidental releases of chemical raw materials and products.

In response to the discovery of off-site migration of these chemicals, and the contamination of off-site water supply wells, the Arsenal acted in cooperation with state and federal regulatory agencies to investigate pathways for groundwater contaminant migration. Interim remedial measures were designed and installed to impede off-post discharge of contaminated groundwaters until contaminant sources could be identified and controlled.Several groundwater barriers were installed at the north and northwest boundaries of the Arsenal, the first of which was at the north boundary barrier system. These projects were pioneering efforts in that the use of slurry wall barriers for control of contaminant migration was relatively new technology. Design and operational experience of such systems was unavailable.

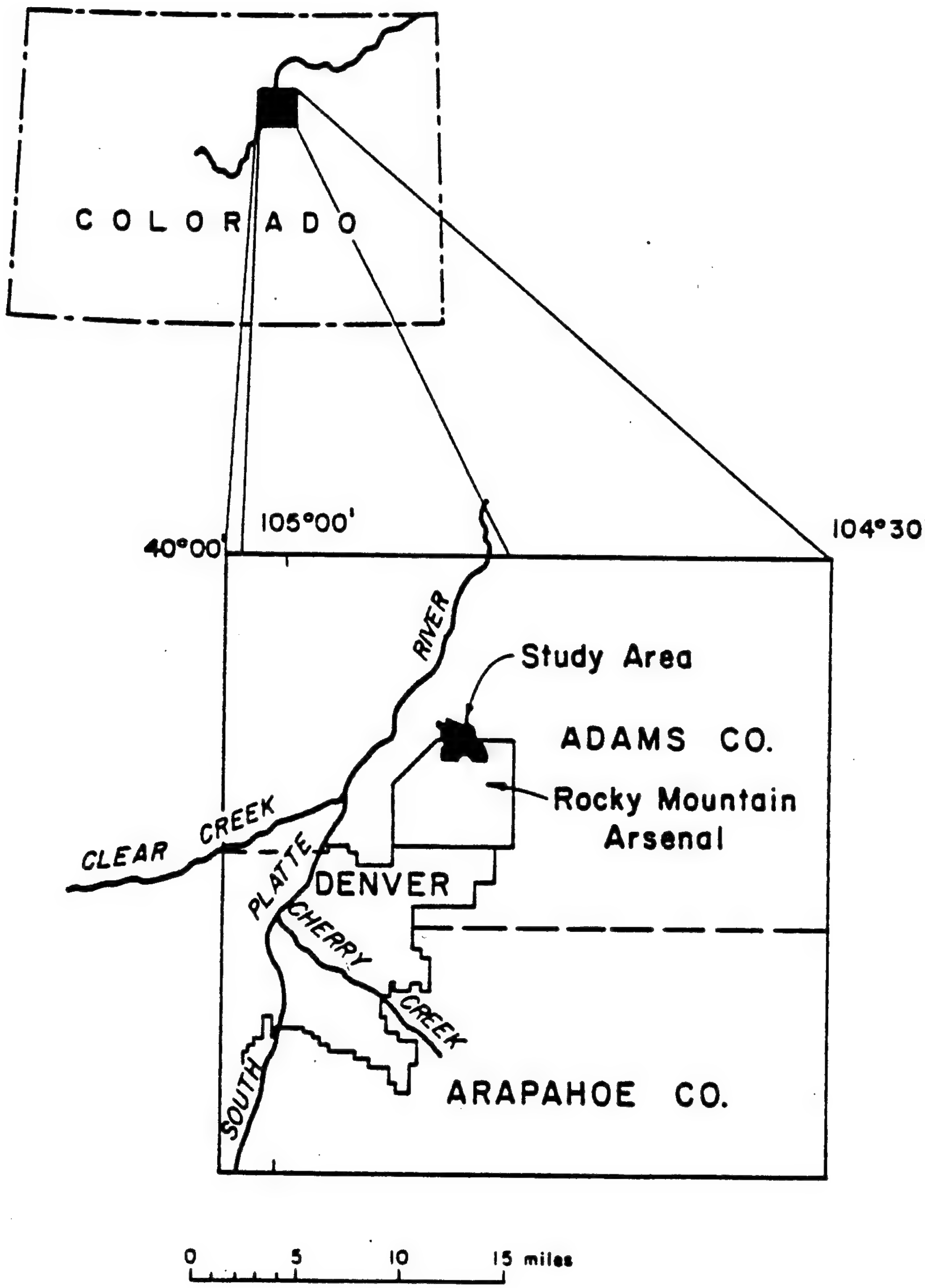


Figure 1.1 Site and Study Location Map

The purpose of this study was to construct, calibrate, and verify a digital groundwater model for use as an operational management and design tool for the north boundary containment treatment system at the Rocky Mountain Arsenal.

The north boundary barrier system was installed to intercept and treat contaminated groundwater flowing to the north boundary of the Arsenal in the surficial alluvial aquifer. The system consists of a bentonite slurry-wall, thirty-five (35) discharge wells, and thirty-eight (38) recharge wells. The discharge wells are utilized to pump contaminated groundwater from the upgradient side of the slurry wall. The water is treated via activated carbon adsorption to reduce contaminant levels to acceptable water quality standards. The treated water is recharged down-gradient of the slurry wall to insure continued quantity of water flowing in the alluvial aquifer north of the Arsenal. In the past, management decisions were made in an action-reaction fashion whereby the present management was based upon past operation practices and their observed results. Barrier system breakdowns due to natural phenomenon and mechanical failures, along with planned maintenance down-time further complicate the operation of the system. The Arsenal undertook this study in order to provide a rational means for planning future barrier operation.

1.2 Study Objectives

The objective of this study was to develop, calibrate, verify, and apply a digital finite element groundwater model (CSU-GWFLOW) to aid the Arsenal in answering operational management and system modification design questions. These questions are concerned with the normal operation of the barrier system at various flow values. Specific questions posed by the

Arsenal were of two basic aims: evaluation of the performance of the existing system, and review of means by which the system could be reconfigured to improve operational flexibility and performance.

One goal of barrier system operation is to minimize the hydraulic gradient across the slurry wall. Bentonite slurry has a hydraulic conductivity in the range of $1x10^{-8}$ to $1x10^{-6}$ cm/sec (USEPA, 1985). Although the hydraulic conductivity of the slurry wall material is quite small, continuity of the wall and key-in of the wall into the underlying formation is not assured, a minimum hydraulic gradient is therefore desirous as an added measure for containment of contaminated groundwaters. Ideally, a reverse hydraulic gradient is desired which would positively prevent advective transport of contaminants through, or under the barrier. Arsenal personnel would like to know which barrier management alternatives would result in a reverse gradient across the slurry-wall or at least which alternatives would result in a minimum gradient across the barrier. Many of the questions posed by the Arsenal were related to this goal.

Constraints to the attainment of a reverse gradient may include:

* Existing discharge and recharge well spacing in relation to each other and to the slurry wall.
* Natural (pre-barrier) flow patterns in relation to placement of the barrier system.

The specific questions posed by the Arsenal include:

(1) What is the natural equilibrium flow rate presently intercepted by the barrier?

(2) What are the rates for pumping and injection wells required to maintain the equilibrium rate, and what gradient across the slurry wall results from the utilization of these rates for the present well configuration?

(3) Is it feasible to achieve a gradient reversal with the existing system configuration?

(4) What is the existing recharge capacity of the aquifer west of building 808 in an area of apparent low transmissivity?

(5) What system alterations, or operational modifications are required to attain the desired reverse gradient?

In addition to the above simulation goals, various barrier breakdown scenarios were investigated.

1.3 Scope of the Study

In order to evaluate the questions posed by the Arsenal, a detailed finite element groundwater flow model was constructed, calibrated, and verified utilizing CSU-GWFLOW (version FEM2D3.1), a code developed by the principal investigator at Colorado State University. For details of the capabilites of GWFLOW and details of the mathematical development of the Galerkin finite element formulation please refer to Warner, 1981 and 1987. This versatile model utilizes triangular elements which allowed detailed definition of barrier system configuration, complex aquifer geometry and hydrologic features such as First Creek.

The construction, calibration, and utilization of the model included the following:

1. Review of local and regional hydrogeology to allow selection of model boundaries, and parameters such as saturated thickness, hydraulic conductivity, and storage coefficient. This consisted of review of existing information compiled as a part of previous studies at the Arsenal and specifically for the vicinity of the north boundary. The information included measurements of potentiometric hand and mapping, test boring and well completion information, pumping and slug-tests results. These data sources are listed in the references. No independant field review was performed for this study other than obtaining routine depth to water measurements.

2. Construction and calibration of the numerical flow model according to the information obtained as outlined above. Limits of the model were selected based upon mapping, test boring, and well completion information. The model was calibrated to pre-barrier potentiometric conditions (February-March 1978). Transient calibration and verification of the model was performed. Transient calibration consisted of refinement of the predictive capabilities of the model utilizing historical barrier operation data for the nine and one quarter year period from start-up of the pilot system to May 1987. The model was tested and refined based upon comparison of model computed versus observed aquifer response to applied stresses of the barrier system.

3. Utilization of the calibrated and verified model to predict the effects of future management decisions and system configuration options. The results of these simulations are discussed in detail in Chapter 4.

4. A quasi three-dimensional version of the model was applied to the North barrier system in the vicinity of the original pilot scale system. This three-dimensional includes both groundwater flow and contaminant transport. This model included both the alluvial aquifer and the upper part of the Denver Formation. It was used to investigate potential migratory pathways between the surficial alluvial aquifer and the underlying Denver Formation.

1.4 Previous Studies

Since the discovery of groundwater contamination in the vicinity of the Rocky Mountain Arsenal in the mid-1950's, a great number of studies have been performed in order to define the complex hydrogeology of the area. Many of the studies are summaries of data obtained from field efforts to characterize the properties of the aquifers at the Arsenal, and the nature and extent of contamination. Numerous studies were performed in the investigation, design, and evaluation of the three boundary containment treatment systems at the Arsenal.

Some of the first reports of aquifer contamination in and around the Rocky Mountain Arsenal were published in 1961. A study by L. R. Petri of the U.S. Geological Survey consisted of a summary of a field investigation to determine the extent of contamination of groundwaters by sodium and chloride containing liquid wastes disposed at the RMA in un-lined basins. This study delineated an approximately 4 and 1/2 square mile plume of saline groundwater moving from the Arsenal to the northwest towards the South Platte River. This study was paralleled by reports by Walton (1961),and Walker (1961) which presented plume delineation based upon the phytotoxic effects of the groundwater when utilized for irrigation.

Studies to determine aquifer characteristics and pathways of migration in the Rocky Mountain Arsenal and surrounding vicinity include Smith and Schnieder (1964), and Konikow (1972). Several studies specific to the north boundary area include: Zebel (1979), May et al (1979), Visipi (1979), and Geraghty and Miller (1979).

Several digital model studies have been performed to simulate the fate of contaminants in the groundwater system in and around the Rocky Mountain Arsenal. The first of these studies was digital simulation of chloride movement in the regional alluvial aquifer at the Arsenal by Konikow (1977). This study was a pioneering utilization of a digital model to simulate transport of contaminants in groundwater. Simulation of chloride movement under natural conditions, and under different remedial schemes of pumping wells for extraction and artificial recharge for dilution was performed.

The model by Konikow was adapted by Robson and Warner (1976 and 1977) to simulate transport of diisopropylmethylphosphonate (DIMP) at the site. Different groundwater interception options were simulated. These model studies were regional models utilizing very coarse grid spacing and could not produce the very detailed results which are the goal of this study. Warner (1979) utilized a more detailed model grid to evaluate groundwater interception options for the north boundary including: bentonite barriers, pumping and recharge wells. Contaminant transport was included in order to estimate the effects of the hypothetical measures on local contaminant levels.

The most recent modeling efforts at the Arsenal included a digital simulation of operation of the northwest boundary system (Warner, Walker, and Ward, 1986). This study was completed in 1986 and the success of the effort was the impetus for the north boundary study and this report. As with

this study, the northwest boundary model utilized CSU-GWFLOW and a very detailed finite element mesh to answer questions concerning the operation of the interception containment system. The code CSU-GWFLOW has been used and documented in several previous studies by Warner (1981 and 1987), Gabaldo-Sancho (1983), Walker (1986),and Nielson (1987).

CHAPTER 2

SITE DESCRIPTION

2.1 Location and General Site Description

The Rocky Mountain Arsenal occupies approximately 27 square miles of land northeast of Denver, Colorado. At present, the site is being studied extensively to identify contamination sources and plan the ultimate remedial activities to bring it into compliance with Federal and State environmental laws and standards. An extensive system of groundwater monitoring has been installed over the entire Arsenal in order to better understand the complex geohydrology of the site. Some remedial activities have been implemented, including installation of three boundary barrier systems, and abandonment of a deep disposal well. Source-area controls such as volume reduction and containerization of Basin "F" liquid are presently underway. Completion of the ultimate site clean-up has been projected to extend well into the next century and cost hundreds of millions of dollars to complete.The site is no longer being utilized for military or private industrial manufacturing.

Use of lands surrounding the RMA varies from agricultural to light and heavy industry. Surrounding land uses are continually evolving due to development pressure of Denver and nearby suburbs.

At present, land use to the south of the site is primarily industrial and residential. Stapleton International Airport is to the south and east of the site. Land use to the west is dominated by residential, commercial, and light industry. To the north and west of the Rocky Mountain Arsenal the land is utilized mainly for agriculture and residences.

Much of the present RMA has reverted to a relatively natural state. The property has a diversity of wildlife and vegetation common to the eastern plains of Colorado. Blue grama shortgrass prairie, woodland, and thicket habitats cover the majority of the site. Game and non-game animals common to the site include mule deer, prairie dogs, jack-rabbits, pheasants, and a wide variety of raptors. A potential use of the site after remediation is a wildlife refuge.

Man-made features on the site include the GB Nerve gas complex, toxic storage area, disposal basins "A" through "F" in the north-central part of the RMA. In the south part of the RMA, the south plants, warehouse and administrative areas are prominent features (Figure 2.1).

2.2 Site History

The Rocky Mountain Arsenal was first utilized for the manufacture and assembly of toxic and incendiary munitions. This activity was the primary site use from the establishment of the post in 1942 to the end of World War II. During the period 1945-1950, the site was utilized to demilitarize obsolete munitions. Explosives, mustard gas, and other agents were destroyed by incineration and detonation. It was during this same period, that parts of the site were first leased to private concerns for the manufacture of pesticides and herbicides.

These private companies included Julius Hyman and Company (1947) and Shell Chemical Company (1952). The private activities were located in the "south plants" area. The Arsenal was utilized to produce and load GB nerve gas agent munitions from 1953 to 1957 (USATHAMA 1983). GB agent munitions were filled at the site until the late 1960's. Until 1957, the wastes generated by government and private sources were discharged into

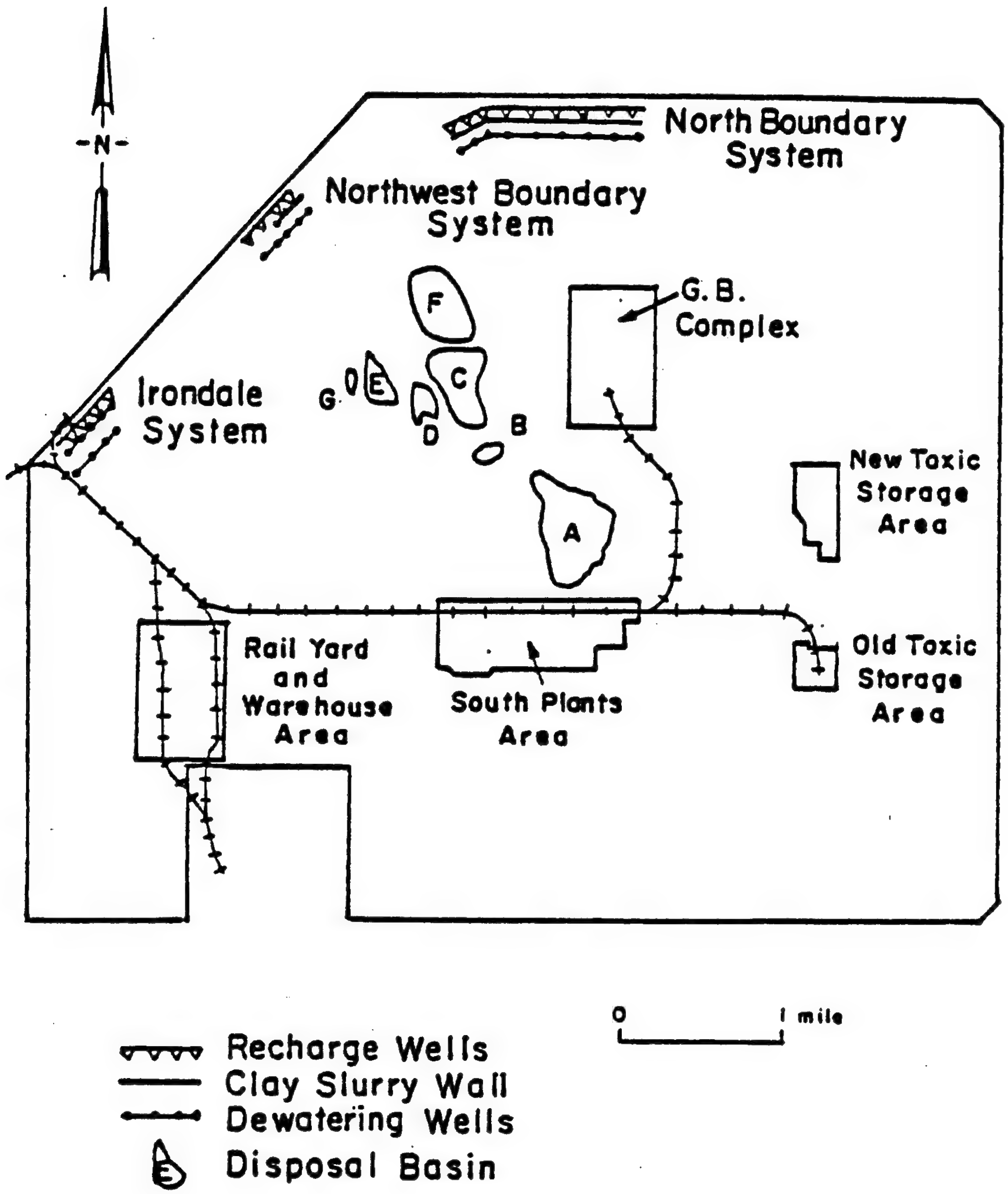


Figure 2.1 Site Layout Map

(from Walker, 1986)

unlined evaporation basins. The basins had been utilized since 1943 for disposal of various chemical wastes and by-products. These disposal practices resulted in contamination of groundwaters over an area in excess of four square miles, from the center of the arsenal site to the South Platte River (Petri 1961, and Walker 1961). In the early 1950's, farmers between the Arsenal and the South Platte River complained of "severe crop damage" due to irrigation with waters pumped from the shallow alluvial aquifer. Several studies (Petri 1961, Walker 1961, and Walton 1961) delineated plumes of "chloride" and "chlorates" over this area by groundwater monitoring and phytotoxic studies.

In response to the complaints, the Army constructed Basin "F", an asphalt-lined impoundment of approximately 93 acres in size. This basin was utilized for further discharge of liquid wastes at the facility. In 1961, a deep (12,000ft.) injection well was constructed north of basin "F". This well was utilized for deep injection disposal of basin "F" fluids. Deep well disposal ceased in the late 1960's after a series of small earthquakes in the Denver area were associated with disposal activities. The deep well was abandoned and officially closed by pressure grouting in 1985.

From 1960 to early 1985, the RMA was utilized for the disposal of chemical and biological munitions. The agents that were processed at the site during this period included: explosive ordinance, mustard gas, anti-crop TX wheat rust agent, and GB nerve gas. Mustard gas and TX agent were disposed of by incineration. GB nerve gas was demilitarized by caustic neutralization and incineration. The GB neutralization process produced large quantities of waste caustic sodium hydroxide solution. In the early 1970's, diisoprophylmethylphosphonate (DIMP), a by-product of GB production was discovered in groundwaters to the north of the Arsenal. In addition,

chloride, dicyclopentadiene (DCPD), and pesticide end-products were identified. DCPD is an organic chemical used in the manufacture of pesticides. In December of 1984, the Colorado State Department of Health identified DIMP in a well near the city of Brighton, Colorado to the north of the Arsenal.

In early 1975, Department of Health issued three Administrative Orders against the Army and Shell Chemical Company. The orders required that the RMA and Shell Chemical Company cease unpermitted discharges of DIMP and DCPD from the property. The orders required that the Army and Shell develop and implement source controls, and install monitoring to determine the extent of pollution and the extent of compliance with the cease and desist orders.(RMA, 1985)

In response to the orders, the Army developed a "Contamination Control Program". The primary goals being definition of the "nature and extent" of the contamination and development of "response actions" to control contaminant migration (Thompson, 1985).

The program identified the major pathways for off-site migration of chemicals in the shallow alluvial aquifer. One pathway that was identified was the North Boundary of the RMA. In 1978, the Army constructed the first of three boundary barrier systems at this location.

In 1981, Shell Chemical Company completed the Irondale barrier system. The Irondale system was named after the community that lies adjacent to the Arsenal at this location (Figure 2.1) The system was installed after the chemical dibromochloropropane (DBCP) was discovered in alluvial water supply wells in the community of Irondale. DBCP, also known as Nemagon, is a nematocide produced at the RMA by Shell (Thompson, 1985).

The Irondale system consists of a two rows of dewatering wells and one row of recharge wells. This system relies on hydraulic gradient control alone, no slurry wall is present at this installation. Total construction cost of the system is reported to be 1.2 million dollars (Thompson, 1985).

The northwest boundary system began operation in October of 1984. It was installed in response to the discovery of a narrow plume of DIMP, DBCP, chloride, and the pesticides dieldrin and endrin. The system consists of half hydraulic gradient control, and half physical boundary. The northwest boundary was constructed at a cost of approximately 5 million dollars. The operation of this system was the subject of a detailed digital model study completed in 1986. For details of the operation of the northwest boundary system, the reader is referred to the report completed by Warner and Walker, 1986.

2.3 Hydrogeology

The Rocky Mountain Arsenal is located within a geologic structural depression commonly referred to as the Denver Basin. The Denver Basin consists of approximately 15,000 feet of sedimentary units including sandstones, shales, and conglomerates. The rock units are overlain by Recent and Pleistocene, Quaternary deposits of alluvial and aeolian origin. The Denver Basin is roughly oval in shape, covering approximately 6,000 square miles. It is hydrogeologically isolated in that the only natural sources of recharge are incident precipitation and stream recharge. Current groundwater withdrawls exceed recharge in the basin. Potentiometric surface declines in the deep sedimentary rocks due to withdrawl exceed 100 to 200 feet in the vicinity of Denver (Geraghty & Miller, 1981).

The surficial geohydrologic units that have been effected by activities at the Arsenalinclude the Quarternary, Recent and Pleistocene alluvial aquifer and the underlying Denver Formation (RMA, 1981).

2.3.a Geology

The Recent and Pleistocene alluvium consists of unconsolidated, stream channel and floodplain terrace deposits. Thickness of saturated alluvium varies over the Arsenal from zero up to 130 feet. Texture of the alluvial materials varies widely from high plasticity silts and clays to very coarse, well graded river gravels. In some areas the alluvial materials may be partially cemented with calcium carbonate (caliche).

This is particularly true in the vicinity of the north boundary where test borings, barrier system wells, and excavation for the slurry wall encountered cemented alluvium (Black and Veatch, 1981). In these areas, permeability of the alluvial deposits is apparently constrained by cementation. The apparent hydraulic conductivities observed in the field are much less than one would predict based on grain size alone. Over much the the Arsenal, the alluvium is overlain by silty-sands and silts of aeolian origin (Figure 2.2). Generally, the alluvium is mapped as a phreatic (water table) aquifer. Areas exist where the overlying aeolian deposits are fine grained, and the potentiometric surface extends to an elevation above the top of the alluvium and the alluvium becomes confined or partially confined. This tendency is found in northern sections 23 and 24 of the arsenal at the north boundary. Results of long-term pumping tests tend to support this conclusion.

In some areas of the RMA, the Tertiary and Cretaceous age Denver Formation outcrops and the alluvium is unsaturated. The Denver Formation

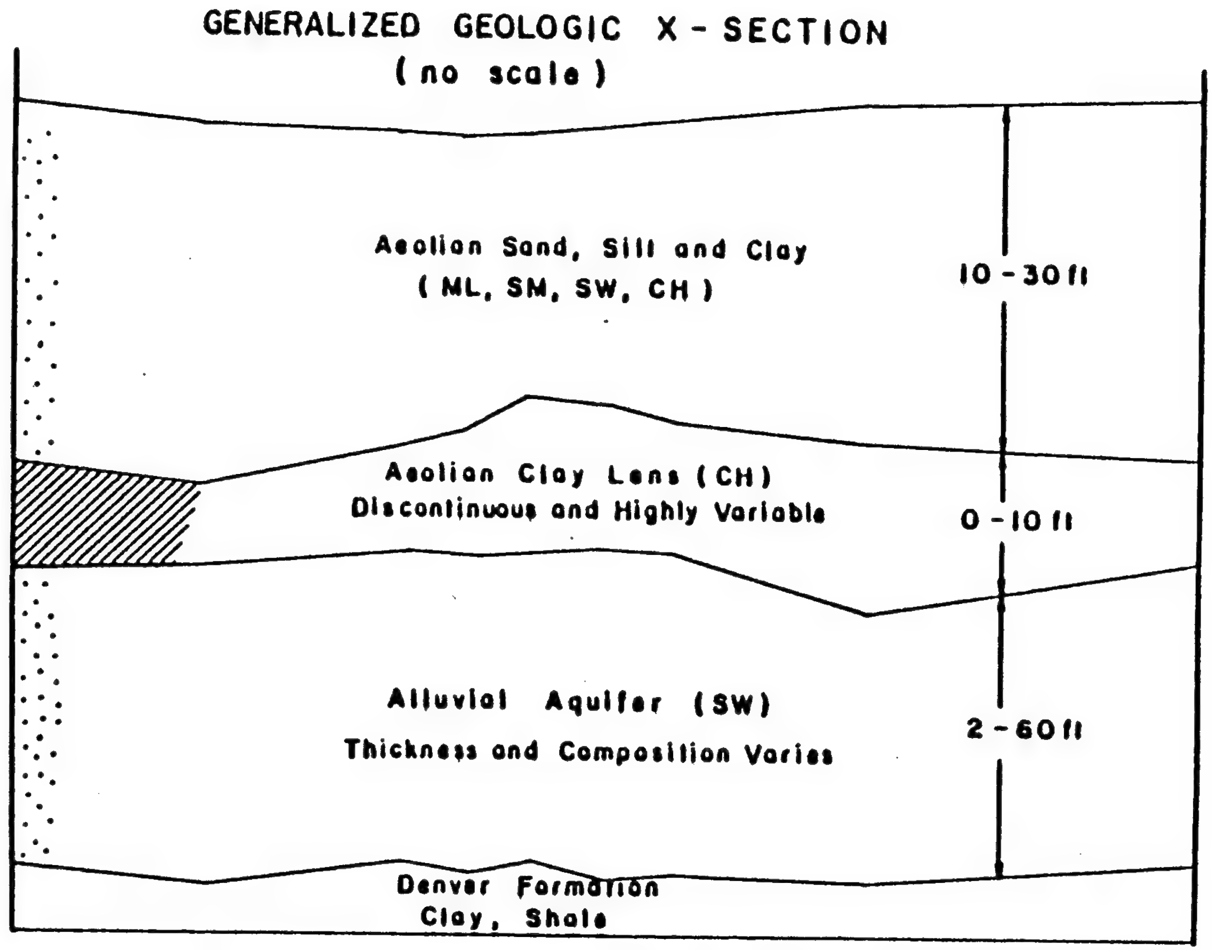


Figure 2.2 Generalized Geologic Cross Section

consists primarily of poorly cemented carbonaceous clay shales. The clay shales are commonly bentonitic. Claystones, conglomerates, with some discontinuous sandstone and siltstone lenses also are considered to be a part of the Denver formation. The Denver formation is a deltaic deposit and therefore has great vertical and lateral variations in texture. Sand lenses in the Denver are thought to be channel deposits of a Cretaceous age delta. The water bearing characteristics of the Denver Formation varies from area to area depending mainly upon textural properties. In general, the matrix hydraulic conductivity of the Denver Formation is much less than that of the overlying alluvium, on the order of 10^{-3} to 10^{-4} cm/sec (0.3 to 10^{-4} ft/day), and flow in the Denver Formation is small as compared to the alluvium (RMA, 1983). Where potentiometric gradients are nearly parallel to the limits of saturated alluvium, flow from or to the Denver is indicated to be absent, or negligible. Areas of the RMA that have been identified as having coarse grained Denver sediments in contact with the alluvium are shown on Figure 2.3. Laboratory and field hydraulic conductivity testing of Denver sands from the vicinity of the north boundary indicate hydraulic conductivities in the range of 10^{-3} to 10^{-4} cm/sec (3 to 0.3ft/day) (May, et.al., 1980). In these areas, flow from the Denver formation to the alluvium may be of some consequence. An example of such an area is between Basin "F" and the eastern limit of section 23. This area was modeled with the assumption that flow from the Denver formation was occurring.

Groundwater flow in the surficial alluvial aquifer is generally to the north and west to the South Platte River and valley fill deposits. Average hydraulic gradient of 30 to 40 feet per mile is indicated by regional potentiometric mapping (Romero and Ward, 1981). Steepest gradients are found

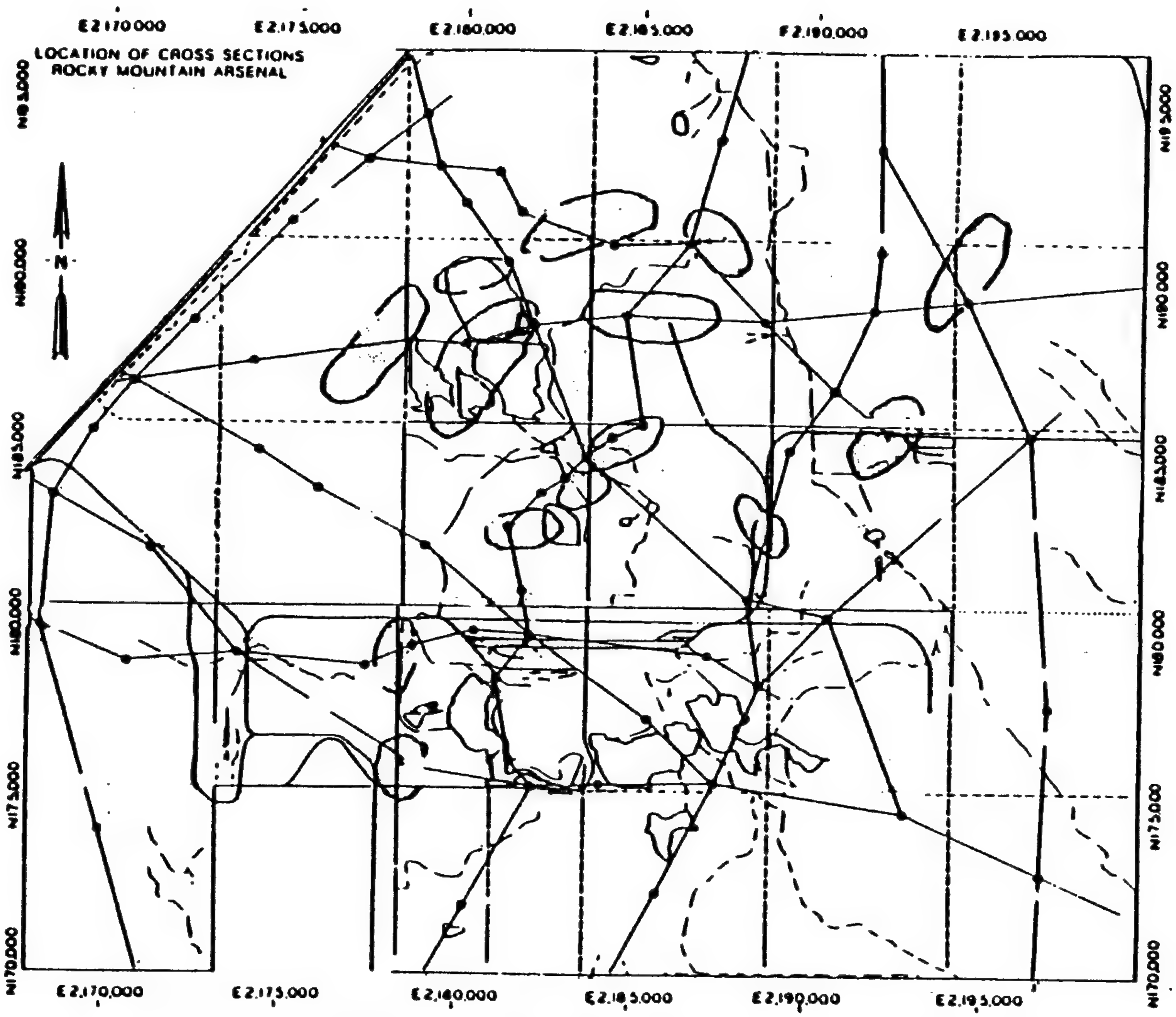


Figure 2.3 Areas of the RMA Where Denver Sands Contact Alluvium
Adapted from RMA, 1983

where the alluvium is unsaturated and flow is in the Denver formation, or where the alluvium is fine grained and/or cemented as at the north boundary at the western limit of the present barrier system. The lowest hydraulic gradients found at the site are in south and central section 23 at the north boundary. This is due to a rapid increase in saturated thickness and hydraulic conductivity north of Basin "F" and the Denver outcrop. In this location, the hydraulic gradient is indicated to be on the order of 5 to 10 feet per mile.

Review of geologic and potentiometric mapping of the RMA indicates that the flow in the surficial alluvial aquifer is largely controlled by paleo-drainage patterns and associated stream channel alluvial depos- its. Deep erosional features in the surface of the Denver Formation result in great lateral variation in transmissivity of the alluvium, both in terms of greatly increased saturated thickness and texture of the deposits. The influence of the erosional features on groundwater flow is further shown on Figure 2.4 which depicts the major components of groundwater flow across the Arsenal. The largest arrows depicting the greatest magnitude of groundwater flux all correspond to erosional features.

2.3.b Hydrology

The Rocky Mountain Arsenal receives an average of 14 inches of precipitation a year. The climate is considered to be arid to semi-arid. The distribution of this precipitation is highly variable, with the majority occurring from March to July. Climatalogic data from nearby Stapleton Airport and Cherry Creek Reservoir indicate that potential evapotranspiration exceeds precipitation by a factor of two to three.

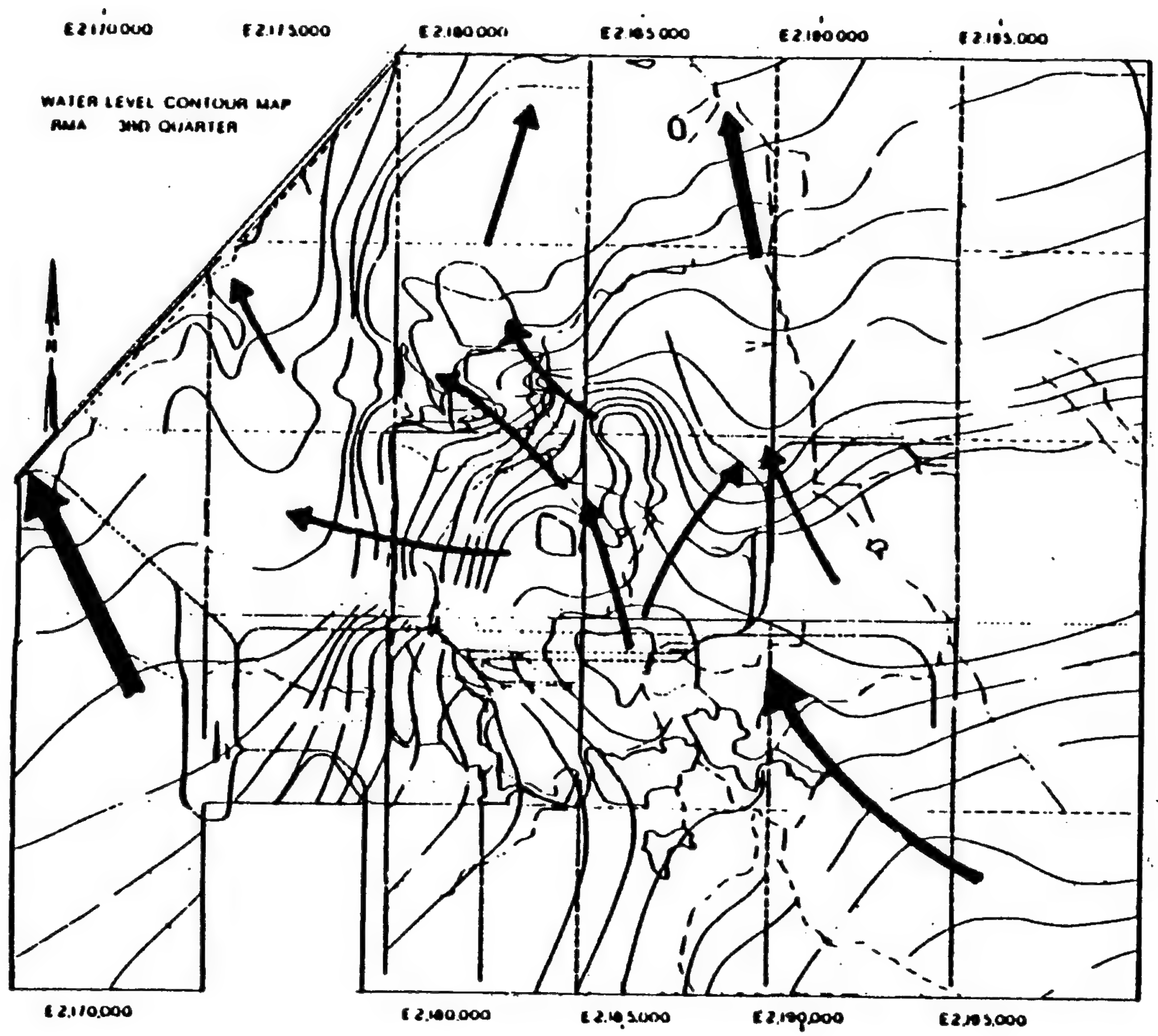


Figure 2.4 Major Groundwater Flow Paths- Surficial Aquifer RMA
Adapted from RMA, 1983

Recharge due to infiltration and deep-percolation of incident precipitation on non-irrigated areas is quite small and is considered neglig- ible on a local scale. In areas of the Arsenal where runoff due to storm events is ponded in natural or man-made depressions with highly permeable surficial soils, (Basins A-D) recharge may be considerable.

Surface drainage at the Arsenal is divided into two primary drainage basins, First Creek, and Irondale Gulch (see Figure 2.5). These surface features are reflections of paleodrainage patterns although somewhat muted by alluvial and aeolian veneers. The entire north boundary model area is within the First Creek watershed, and for this reason, discussion of surface hydrology will be limited to First Creek.

First Creek is an intermittent stream that enters the site approximately 3/4 of a mile north of the southeast corner of the RMA. It leaves the site at the north boundary. First Creek drains poorly vegetated, short grass prarie lands. Convective thunderstorms in the spring and summer can lead to flash flood events which have in the past, caused some operational problems at the north boundary for wells located directly on it's overbank area. However, the Army has made structural changes in this area to prevent this from happening. A typical low-flow for First Creek is in the range of 0.0 to 0.3 cubic feet per second.

In sections 19 and 24, First Creek is fringed by marshy areas with cattail and other wetlands vegetation. It is apparently based upon the vegetation and depth to groundwater measurements, that this area is a groundwater discharge zone. The discharge/recharge relationship for First Creek in other areas is unclear. Three stream gauging stations are located on First Creek. Two stations are located on the RMA property, one at the eastern boundary of the site, and the other at the north boundary. The short

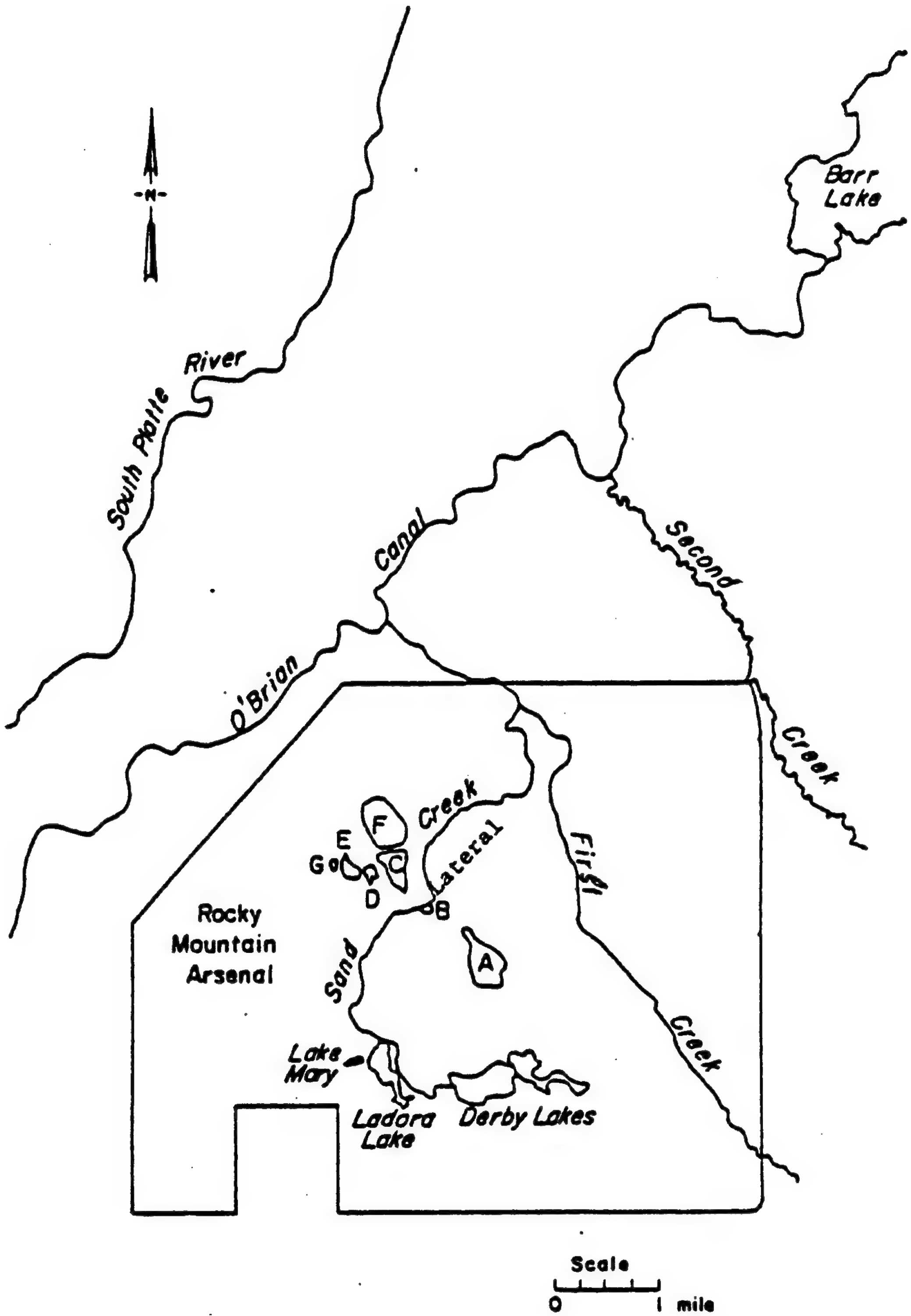


Figure 2.5 Surface Hydrologic Features RMA and Vicinity
From Walker, 1986

period of record available for these gauges indicates that there is a net loss in streamflow across the site during the spring and early summer months when there is continuous flow in the channel (Resource Consultants, 1984). What percentage of the loss is due to evapotranspiration is unclear. The third gauge on First Creek is located adjacent to Route 2, approximately one half mile north of east 96th avenue and the Arsenal boundary. This gauge is located just upstream of the confluence of First Creek and O'Brian Canal. Data for this gauging station was unavailable at the time of this report.

A five acre surface impoundment is located on First Creek, approximately one-third mile north of the Arsenal north boundary. The water level in this impoundment annually fluctuates several feet according to the balance of inflow from First Creek versus evaporation and leakage losses. It is thought that the pond is in hydraulic contact with the aquifer, but somewhat constrained by fine grained sediments on the bottom.

A bog is located at the RMA north boundary between the barrier system and 96th avenue. The bog is approximately two acres in size was present prior to installation of the slurry wall barrier system. Pre-barrier potentiometric mapping indicated that this bog is a natural groundwater discharge zone. Treated groundwaters are currently being discharged to the bog in order to augment the north barrier recharge capability.

2.4 The North Boundary Containment Treatment System

As previously indicated in Section 2.2, a pilot boundary barrier system was installed at the north boundary in July of 1978. Diisopropylmethylphosphonate (DIMP) and other organic chemical contaminants had been detected in groundwaters off-post to the north in 1974. DIMP had been detected in water supply wells belonging to the City of Brighton

Colorado. The Colorado State Department of Health issued three Cease and Desist Orders in April of 1975 that required the Army to stop surface and subsurface discharge of DIMP and DBCP from the site.

The pilot system operated until 1981, when the barrier was expanded to it's present configuration.

2.4.a Pilot Containment Treatment System

The pilot containment treatment system consisted of a 1500 feet long bentonite slurry wall, six dewatering wells, and thirteen recharge wells (Figure 2.6).Contaminated groundwater was pumped from the alluvium upgradient of the slurry wall, treated by granular activated carbon adsorption, and recharged downgradient of the slurry wall. It was designed to intercept a plume of DIMP flowing across the north boundary from the vicinity of Basin "F". The design and construction of the pilot system was a pioneering effort in that application of a slurry wall to contaminant migration interception was quite new at that time. Slurry wall technology had been utilized primarily for seepage control in earthen dams up until this time.

The slurry wall was 30 inches in width and excavated to an average depth of 25 ft. The bottom two feet were keyed into the underlying Denver formation. The dewatering wells were installed approximately 225 feet upgradient of the barrier and were spaced every 225 feet along the length of the barrier. They consisted of 8-inch diameter casings in 30-inch, gravel packed, borings.

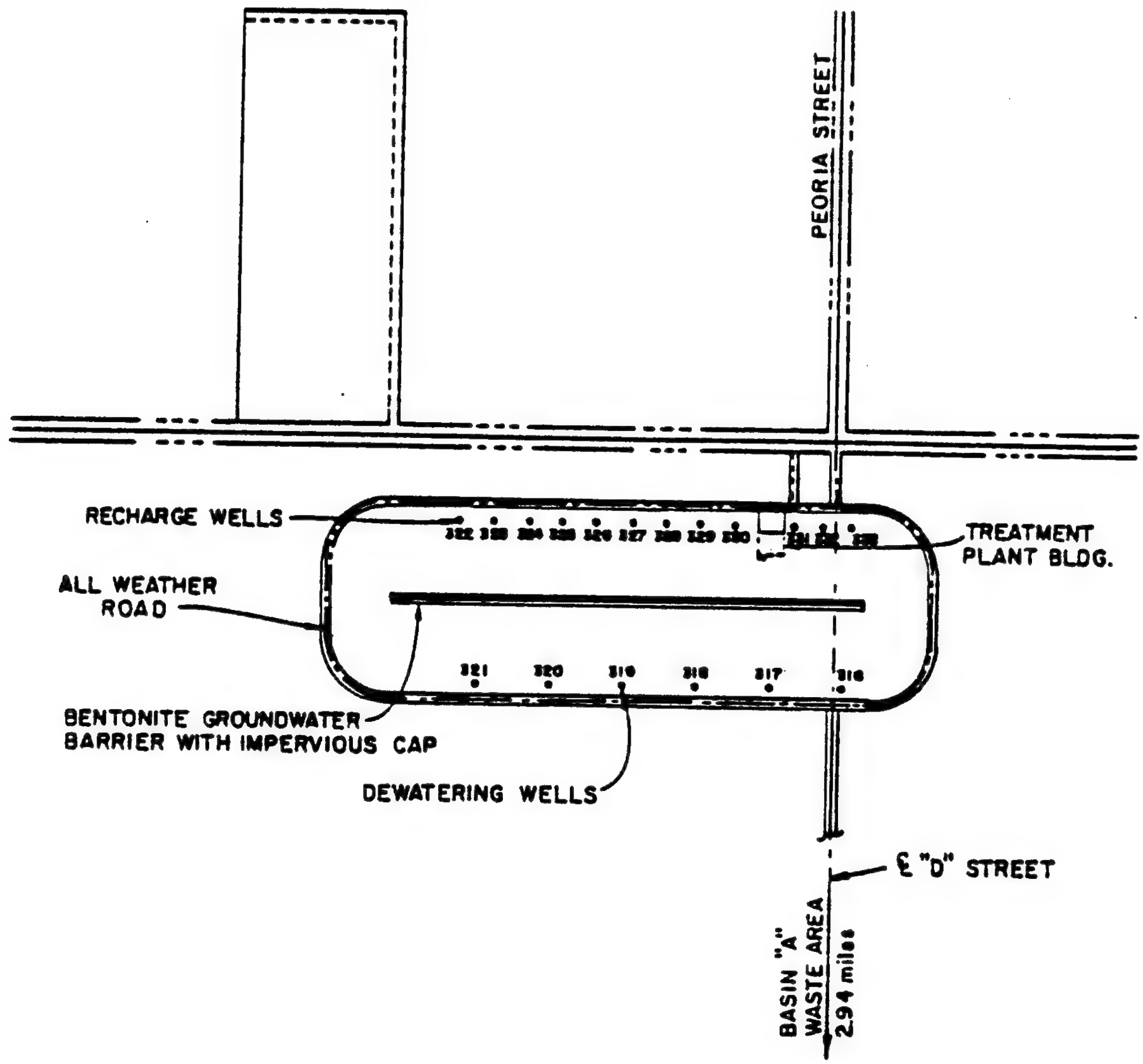


Figure 2.6 Pilot System Layout
Scale 1"=300 feet

Notes:

1. Pilot system discharge wells 321, 320, 319, 318, 317, and 316, correspond to full-barrier wells 301, 302, 303, 304, 305, and 306.

2. Pilot system recharge wells 322, 323, 324, 325, 327, 328, 329, 330, 331, 332, and 333 correspond to full-barrier wells 401 through 412.

Recharge wells were completed by installing 18-inch diameter casings in 36-inch diameter holes with gravel packing. Recharge wells were spaced approximately every 100 feet downgradient. Both recharge and discharge wells were screened over the entire saturated thickness. Total construction cost of the pilot system was $490,000, not including the cost of the treatment system which was leased.

Detailed operations data for the pilot system was recorded for the first year of operation. This information was summarized in a series of reports entitled "Pilot System Monthly Update,(Jan, Feb, Mar, and April, 1979"). The system operated at an average of 50 gallons per minute over this period. In the late 1970's, the Arsenal decided to extend the barrier system to it's present length. The need for the barrier expansion was identified as more observation data became available and water quality standards evolved for DIMP and DBCP.

2.4.b The Present System

Expansion of the pilot system to the present length included the following: (RMA, 1985)

1. Extension of the slurry wall 3,840 feet to the east and 1,400 feet to the west. The extensions were tied into the pilot system slurry wall and were constructed in the same manner as the pilot system. Total length of the slurry wall after expansion is reported to be 6,740 feet.
2. Installation of 29 additional dewatering wells in the alluvium,for a total of 35 dewatering wells.
3. Installation of 26 additional recharge wells, bringing the total to 38.

4. Installation of 19 Denver Sand dewatering wells.

5. Expansion of the treatment system to allow treatment of up to 600 gallons per minute. The leased treatment system was replaced with units which the Army presently owns.

Construction was completed and the system was operational in late 1981. Total cost of construction was approximately 6 million dollars (Thompson, 1985). Layout of the North boundary barrier system is shown on Figure 2.7.

The bentonite slurry wall was constructed by excavating a vertical trench approximately 30 inches wide. A typical trench section is shown on Figure 2.8. A bentonite clay slurry was added as the excavation progressed to hydraulically shore the sides of the excavation. The trench was backfilled with soil blended with bentonite slurry. The trench was to be excavated a minimum of two feet into the Denver formation. Because the depth of excavation required to reach the Denver formation was sometimes quite deep, soil was stripped along the barrier alignment to a pre-determined "working surface" depth. Total depth of excavation from the working surface averaged about 20 feet, with a maximum of slightly in excess of 40 feet. A trapezoidal cover cap was constructed above the slurry wall in order to prevent drying and mechanical deformation of the bentonite backfill.

The alluvial dewatering wells consist of 6-inch steel casing, gravel packed in 16-inch diameter auger holes. The new recharge wells consist of 12-inch casing in 24-inch gravel packed holes. Each dewatering well has high and low-water probes which control pump on and off levels. Likewise, each recharge well has probe activated valves which shut off flow to the well when water levels in the casing reach a point one and one half feet below the floor slab of the well housing (Figures 2.9 and 2.10).

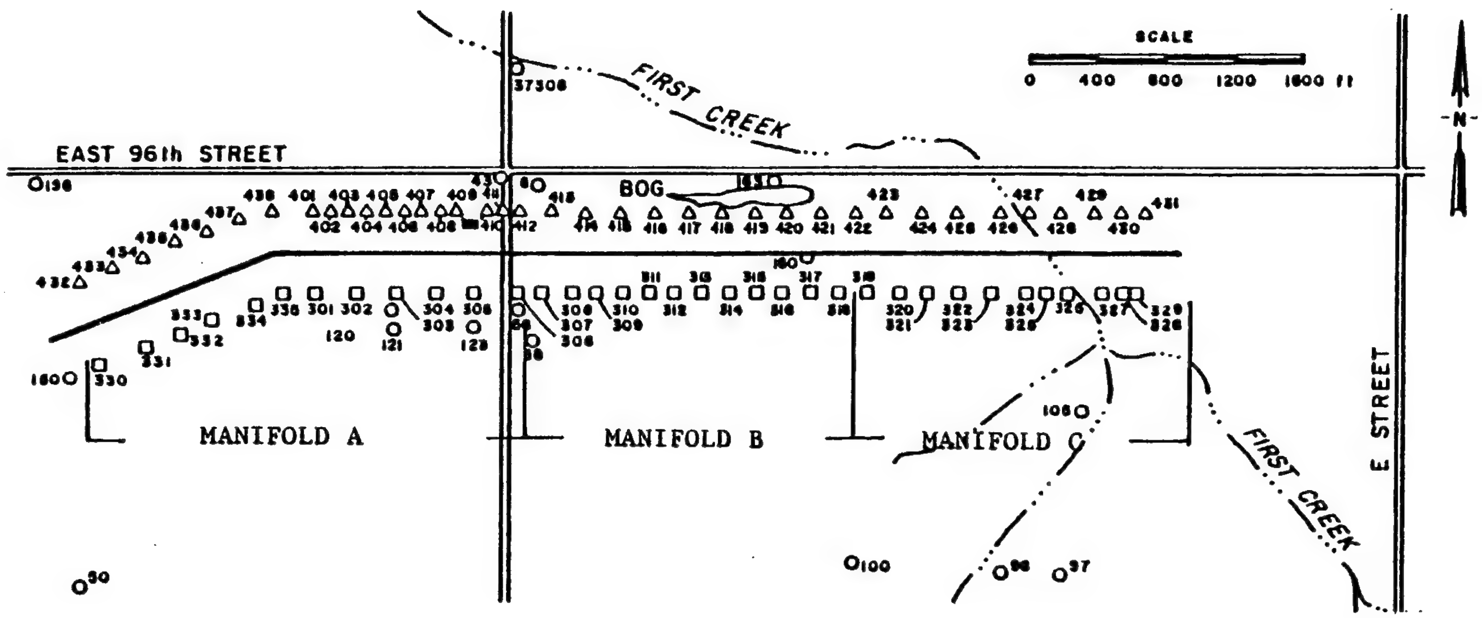


Figure 2.7 North Boundary Containment Treatment System Layout
Legend:
Monitoring Wells = Circles, Discharge Wells = Squares,
Recharge Wells = Triangles

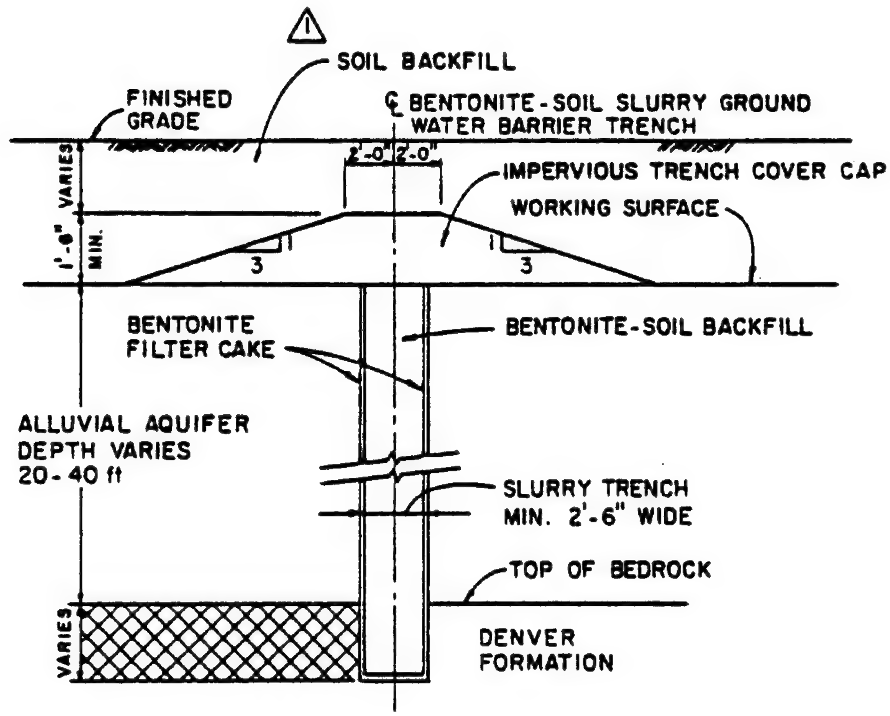


Figure 2.8 Slurry Wall Detail
(from Black and Veatch, 1981)

(Not to Scale)

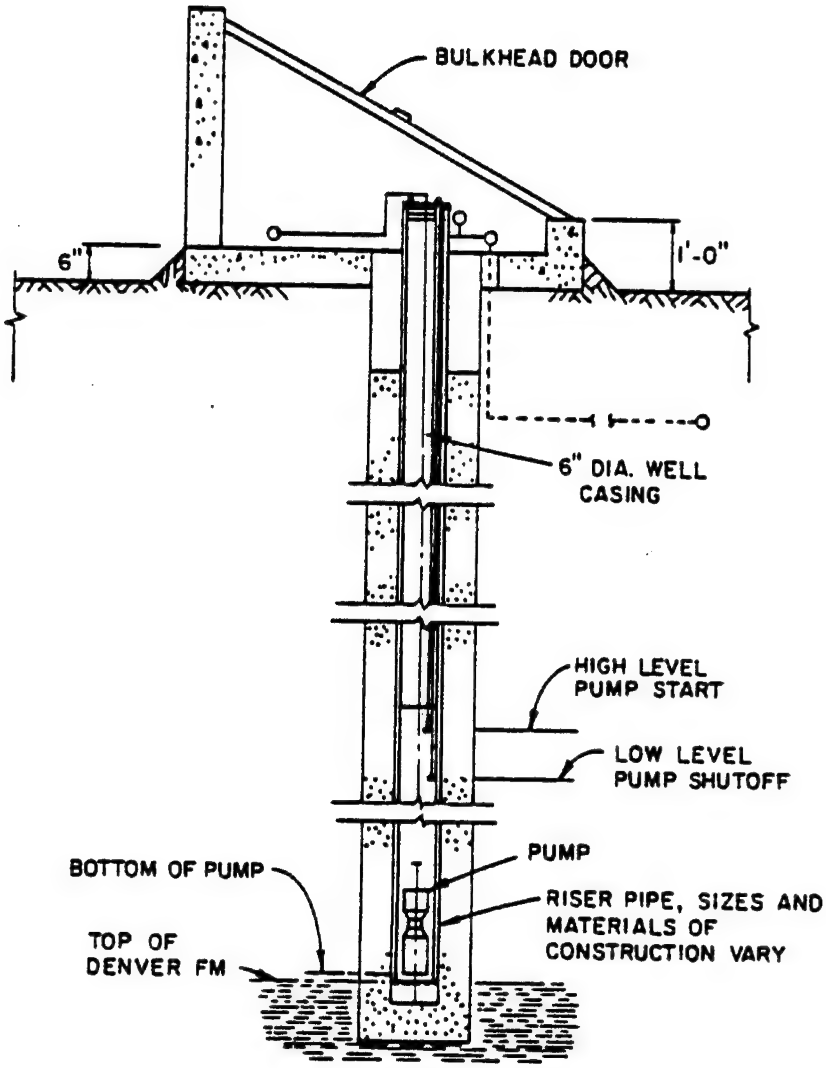


Figure 2.9 Dewatering Well Detail
(from Black and Veatch, 1981)

Notes:

1. Wells screened with 6-inch diameter, 0.060 continuous slot 316L stainless steel, length varies from 3 to 13 feet, average of 7 feet.

2. Each well equipped with a 6-inch diameter, 4 feet. long sand trap located below the screened section. steel casing w/end cap.

3. For details of pump settings and piping, see "Liquid Waste Disposal Facility, North Boundary Expansion", Design and As-built Drawings, Black and Veatch, Kansas City, Missouri, 1982, 90 sheets.

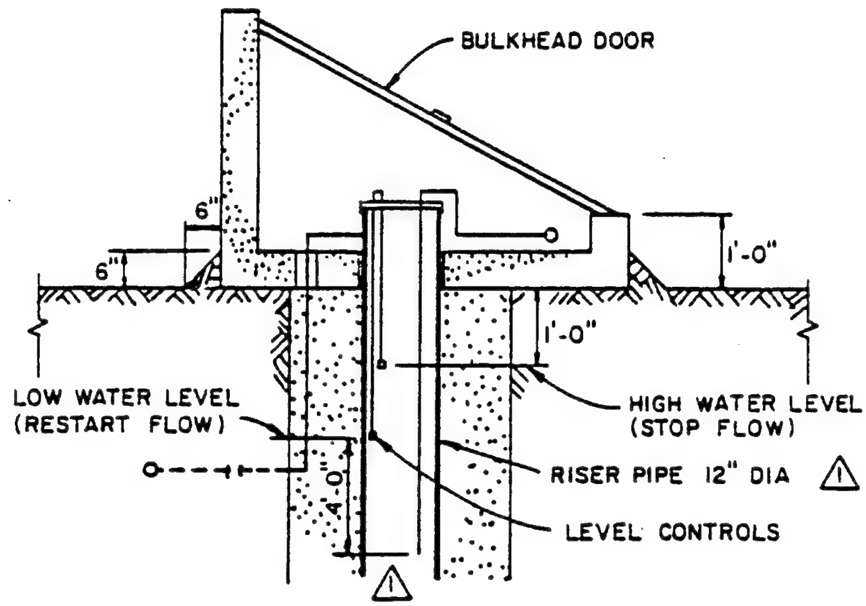


Figure 2.10 Recharge Well Detail
(from Black and Veatch, 1981)

Notes:

1. Wells screened with 12-inch diameter 0.060 continuous slot 304 stainless steel screen, length varies from 5 to 16 feet with an average of 10.5 feet.

2. Each well equipped with a 12-inch diameter, 4 feet long sand trap located below the screened section, consists of steel casing w/end cap.

3. High Water Level (stop flow) setting typically an elevation 1.5 feet below top of well housing floor slab. Low water (restart flow) level typically 8-10 feet below High Water Level. Present float settings may differ from design (Sweter, Ward, 1987).

4. For details of piping and design float settings, see "Liquid Waste Disposal Facility, North Boundary Expansion", Design and As-built Drawings, Black and Veatch, Kansas City, Missouri, 1982, 90 sheets.

A total of nineteen dewatering wells were completed in sands of the Denver formation. These wells were installed after contamination was detected in these units. The Denver formation wells were only utilized for a very short time at the north boundary. These wells were not included in the model and therefore will not be discussed any further.

The north boundary barrier system dewatering wells are divided into three collection manifolds. The three manifolds, A, B, and C, are discharged to separate flow equalization sumps and are treated by separate carbon adsorber units (Figure 2.11). Each of the manifolds intercept groundwater of different quality. Manifold A includes dewatering wells 301 through 306 and 330 through 335 on the west end of the barrier. This manifold primarily intercepts a plume of DIMP flowing from the Basin "F" area. Manifold B includes wells 307 through 318 and intercepts a DBCP plume. Manifold C is located on the eastern side of the barrier and includes wells 319 through 329. Manifold C intercepts low concentrations of DIMP, presumably from the south plants area. In addition to the DIMP and DBCP plumes, the system intercepts measurable concentrations of other organics including: Dicylopentadiene (DCPD), chlorinated pesticides- aldrin, dieldrin, and endrin, and several organo-sulfur compounds. Inorganic contaminants include chloride and fluoride. The inorganic ions are not removed by activated carbon adsorption. The layout of pumping wells according to each manifold is shown in figure 2.7.

Each manifold discharges to a separate influent wetwell. The wetwells are equipped with level sensing probes which control operation of the manifold wells according to the balance of inflow, and outflow through the adsorbers. The wetwell discharge is pumped through pre-filters to remove sediments

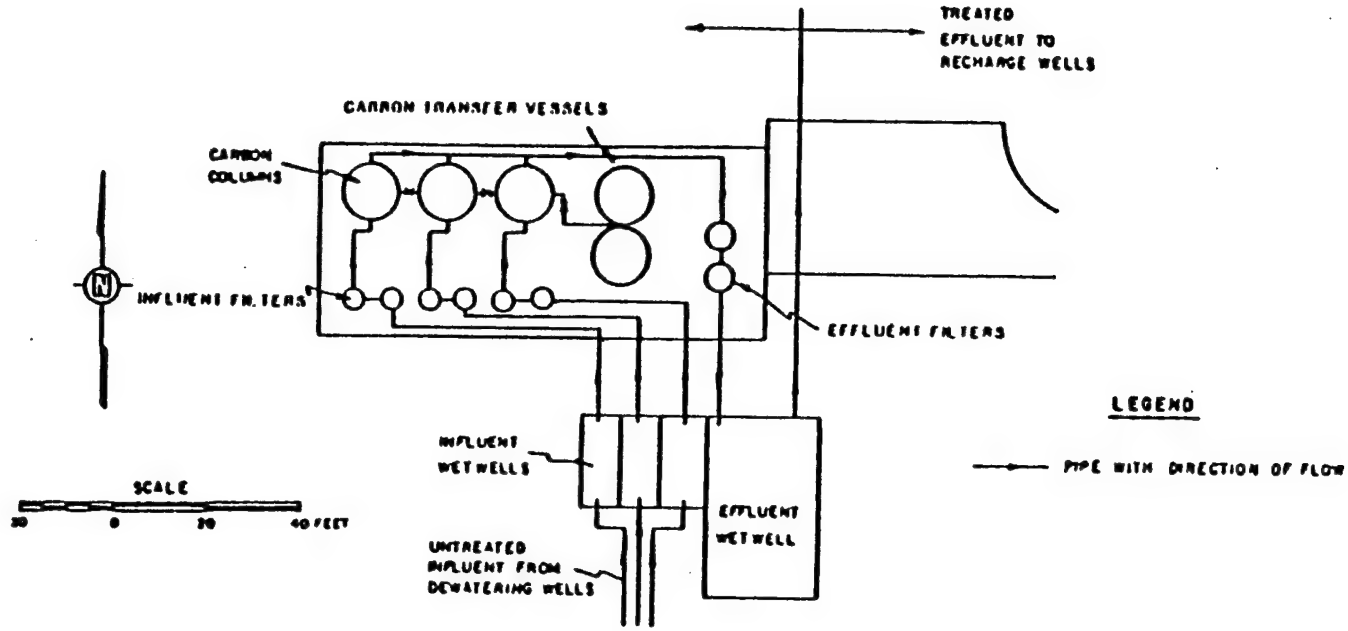


Figure 2.11 System Schematic
(from RMA, 1985)

extracted with the groundwater. The filtered water is passed up through the pulsed-bed carbon adsorber columns. Discharge from each manifold is treated separately because of the very different contaminant distribution across the barrier.

Efficiency of the adsorption process is directly proportional to concentration of the incoming effluent. If the entire barrier discharge was mixed before treatment, dilution of the individual contaminants would occur and less mass of contaminant would be removed per volume of water treated due to a reduction in adsorption potential. Cross treatment of the manifold flows is possible through cross valving the outflow from the influent wetwells. Cross treatment is only performed in emergency situations when one manifold system must be taken off-line for repairs.

Discharge from the carbon adsorbers is post-filtered to remove carbon fines that have been picked up in the treatment process. This step is very important as carbon fines have tended to clog the recharge wells, greatly limiting recharge capacity. The filtered adsorber outflows are discharged to a common effluent wetwell. The treated and blended water is pumped to a common distribution manifold. Recharge to each individual well is a function of pressure in the distribution manifold at the well, well screen efficiency, and aquifer properties.

2.4.c System Operation History and Objectives

Minute by minute operation of the barrier system is quite complex. Controls to flow through the system are located at several points along the process. Float activated pump switches control the activity of each individual discharge and recharge well. Each influent wetwell has level sensing probe switches that prevent overfilling of the wetwells by shutting down the respective manifold. The pumps which transfer water from the

influent wetwells through the carbon adsorbers are level probe controlled. Level activated controls on the effluent wetwell prevent overfilling of this reservoir. Flow through the system at any time may be controlled by the balance of inflow and outflow to each component.External restrictions to system operation include aquifer properties (eg. transmissivity, storage coefficient), and the natural equilibrium flux to the boundary system.

In normal operation of the barrier system, the operator selects desired adsorber flow-through rates and interactively adjusts valves between the influent wetwells and the adsorbers to approach or attain the target flow rates. The other components of the system adjust through changes in reservoir storage until the limiting component controls the average operation of the barrier. If recharge capacity is the controlling factor, the system is controlled by the level probe switches in the effluent sump and the other components of the system adjust. Flow through the system is in a continuous dynamic state. Average flow through the system recorded via totallizing flow meters is actually a time-weighted average that may reflect several on-off cycles of one or more components of the system.

Observations of system operation and discussions with RMA personnel indicate that manifolds typically cycle on and off as much as several times per hour. Individual discharge wells located on the west end of manifold A may not pump for several consecutive manifold cycles due to slow aquifer response in this area.Individual wells on the recharge and discharge side of the barrier may be inoperable at any time due to electrical and mechanical failures.

Operation of the barrier system has experienced mechanical, equipment problems during the years of operation from 1978 to present. Problems include lightning strikes, freezing and component wear and tear. The system is located in short grass prairie and consequently is quite exposed to the

elements. Well and manifold piping were well insulated. Each well is unit contains pump controls, valves, and flowmeters, these components have been susceptible to freezing and breakage (Thompson, 1985). Since 1985, modifications to improve plant operations have been completed which has minimized the problems associated with weather and equipment.

Research on methods of artificial recharge have determined that long term recharge capacity is highly dependant upon quality of incoming water (Asano, 1985). Suspended solids can quickly clog the well screens and formation, greatly reducing the available recharge capacity. Logically, the finer grained the formation is, the more susceptible to clogging. The north boundary system is no exception.

Average flow through the system is indicated to be in the range of 200,000 to 500,000 gallons per day. With this very large volume of water being treated, even minute quantities of suspended matter can cause clogging problems over the life of the system. Wells screened in finer grained portions of the alluvium appear to have greater clogging problems. Recharge efficiency can be maintained by frequent re-development of the wells. Well development efforts may only effect the screen, gravel pack, and a very small portion of the surrounding formation, and rarely can the original well capacity be attained.

Recharge well efficiency is also effected by: biological action, chemical impurities, dissolved gasses, and entrained air (Asano, 1985). At the north boundary, clogging due to activated carbon fines is one of the biggest problems and is currently being addressed. Ideally, well screen and gravel pack losses should be small. Comparison of observed potentiometric surface levels in the aquifer with "valve off levels" of recharge wells indicates that in some cases, well losses may exceed 20 or more feet.

With all of the previously mentioned complications to system operation, it is not surprising that barrier operation rates show a great deal of fluctuation on seasonal, monthly, weekly, and daily time frames. Seasonal fluctuations can be attributed to cold weather operational problems and dynamism of the aquifer system due to natural variations in interaction with First Creek and the bog. Monthly, weekly, and daily, fluctuations may be due to cold weather operation problems and the general dynamic nature of the containment treatment system.

Long-term operation of the barrier system at rates much greater, or lower, than natural flux to the barrier is not possible. The system is self-regulating in that overpumping would desaturate the aquifer and limit extraction efficiency, while underpumping would allow water levels to rise and increase pumping efficiency. This self regulation has been observed for the historical system operation. Regular annual cycles of underpumping during the winter months lead to high groundwater conditions in the spring, and allow several months of overpumping. RMA personnel have typically attributed the spring high groundwater conditions to increased flow in the surficial aquifer in late winter and early spring. The results of modeling indicate that high groundwater conditions are more likely a result of underpumping of the barrier system during winter months.

The present barrier system has been in operation for about six years. Approximately 32 months of operations data were available for review and use in transient calibration and verification. Table 2.1 summarizes the range and averages of the thirty-two months of data.

Figure 2.12 depicts the monthly average total barrier pumping data of record. These averages include twelve months of data for fiscal year 1984, twelve months of data for fiscal year 1986, and eight months of fiscal year 1987.

Table 2.1 Summary of Monthly Average Barrier Operations Data

Manifold	Range	Average
A	27 - 132gpm	61gpm
B	26 - 129gpm	77gpm
C	32 - 142gpm	94gpm
Total	124 - 360gpm	232gpm

For comparison with Table 2.1, the monthly average pumping rates for the period (February to May 1987) are 35, 60, and 116 gallons per minute for manifolds A, B, and C respectively. It is felt that however successful this pumping scheme is in achieving its goals, it may lead to a shifting of flow paths of other contaminant plumes along the barrier.

Since January 1987, the Arsenal has been operating the barrier system with the following goals in mind.

1. To maintain lower water table elevations in the area near Manifold C as a preventative measure against barrier overflow.
2. To control fluoride concentration in the effluent by adjusting flow ratios of manifolds A, B and C.

Arsenal personnel have indicated that if manifold C is pumped at approximately 100 GPM then both of the above goals can be satisfied. Under these operating conditions the threat of water overflowing the Barrier is not as eminent, and high fluoride concentrations intercepted by manifolds A and B are diluted in the effluent by relatively clean water from manifold C.

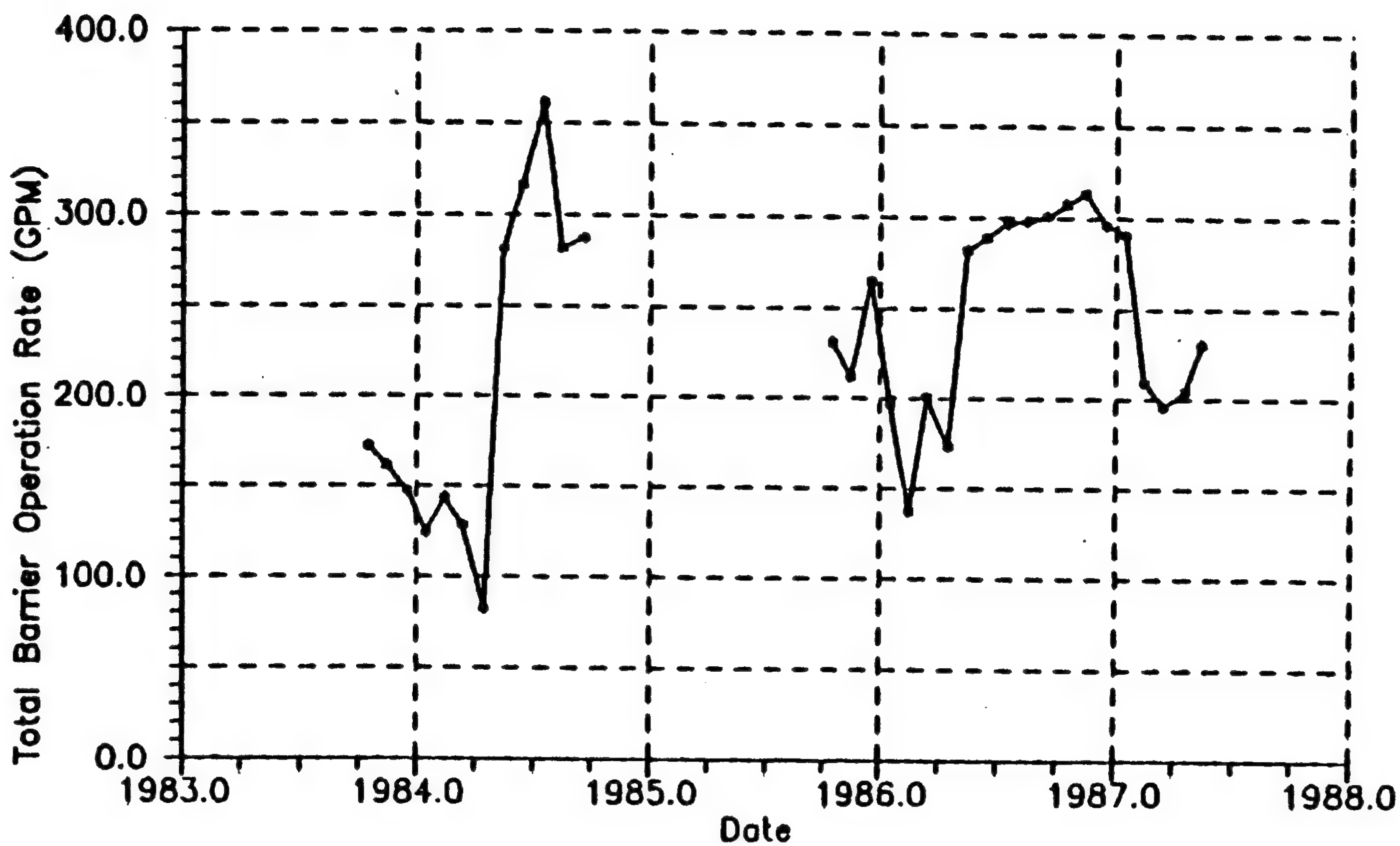


Figure 2.12 Monthly Average Barrier Operation Rates
(Sum of all Manifolds)

CHAPTER 3

MODELING PROCEDURE

The first step in development and application of the model CSU-GWFLOW to the North Boundary Barrier System was evaluation of existing data sources and definition of the aquifer properties, geometry and boundary conditions. The end product of this step was construction of the finite element mesh and an initial data set for calibration. Calibration consisted of refinement of the model input parameters in an interactive fashion in order to develop the predictive capability of the model. The model parameters were adjusted in accordance with what field data were available until the best approximation of the observed potentiometric surface levels were obtained by the model. Calibration involved both steady-state and transient simulation of observed aquifer responses. After the predictive capabilites of the model were verified then the model was used to evaluate operational management alternatives for the barrier system.

3.1 Program CSU-GWFLOW

Program CSU-GWFLOW is a Fortran 77, finite element computer code that is capable of solving 2D groundwater flow problems. The program was developed by Dr. James W. Warner as a part of a groundwater flow and solute transport modeling package. For detailed development of the model, please refer to Warner (1981 and 1987).

CSU-GWFLOW is a highly versatile and adaptable model. It is capable of simulating steady and unsteady; confined and unconfined areal flow of groundwater. Model capabilities include spatially variable: aquifer

properties, initial, and boundary conditions. Point and distributed recharge stresses can be specified. The model utilizes triangular elements and linear shape functions.

3.1.a Brief Overview of the Model Development

The equation describing transient, two-dimensional areal flow in a confined, hetergeneous isotropic aquifer is shown below (equation 1, McWhorter and Sunada, 1977)).

$$\frac{\partial}{\partial x}(T\,\frac{\partial h}{\partial x}) + \frac{\partial}{\partial y}(T\,\frac{\partial}{\partial y}) = S\,\frac{\partial h}{\partial t} + W + \sum_{p=1}^{m} Q_p[\delta(x-x_p)\ \delta(y-y_p)] \tag{1}$$

Where:

$T = T(x,y)$ = transmissivity, $[L^2/T]$,

$h = h(x,y)$ = potentiometric head, $[L]$,

$S = S(x,y)$ = storage coefficient (dimensionless)

$W = W(x,y,t)$ = diffuse vertical recharge, $[L/T]$,

$Q_p = Q_p(t)$ = volumetric water flux at a point located at (x_p, y_p), $[L^3/T]$

$\delta(x-x_p)\ \delta(y-y_p)$ = Dirac-Delta function at point x_p, y_p,

The model CSU-GWFLOW solves equation 1 using the Galerkin finite element method of weighted residuals. For the phreatic (water table) case the general partial differential equation is non-linear in $h(x,y,t)$. For the unconfined case the transmissivity (T), in equation 1 above would be replaced by the product of hydraulic conductivity (K) and the thickness of

flow (h). For this case, CSU-GWFLOW solves equation 1 in a piece-wise quasi-nonlinear fashion whereby transmissivities are updated at the end of each time-step to reflect changes in head computed for the time step.

The Galerkin finite element method of weighted residuals reduces the linear partial differential equation to be solved into a set of matrix equations. The area to be modeled is subdivided into discrete elements (the model CSU-GWFLOW uses triangular elements). Element matrices are developed to express the variable properties of each element. The element matrices are combined onto "global matricies" that form a set of algabraic equations that describe the entire domain of interest. The set of algebraic equations includes the boundary condition of the domain. The set of equations is then solved numerically to obtain the solution for potentiometric head h.

The Galerkin finite element method is applicable to equations that can be expressed as a continuum over a domain of interest and as a linear differential operator $L(u) - f = 0$. In the case of CSU-GWFLOW:

$$L(h) = \frac{\partial}{\partial x}\left(T \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T \frac{\partial}{\partial y}\right) - S \frac{\partial h}{\partial t} + W + \sum_{p=1}^{m} Q_p[\delta(x-x_p)\ \delta(y-y_p)] = 0 \quad (2)$$

The function $h(x,y,t)$ is approximated by a trial function $\hat{h}$ which is of the form:

$$h(x,y,t) \approx \hat{h}(x,y,t) = \sum_{j=1}^{n} G_j(t)\ \phi_j(x,y) \quad (n = \text{the number of nodes})$$

where:

h = actual function

$\hat{h}$ = the trial function

G_j= an unknown coefficient ($\hat{h}_j$ in the case of CSU-GWFLOW)

ϕ_j= a linear independant basis, or shape function

The trial function is substituted into the linear operator (eq 2)

$$L(\hat{h}) = L[\sum_{J=1}^{n} \hat{h}_j \, \phi_j] = R \qquad (3)$$

Substitution of the trial function $\hat{h}$ into linear differential operator L results in a residual R. The method of weighted residuals refers to a method by which the unknown coefficients in the approximation of $\hat{h}$ are determined by minimizing the residual R. With the Galerkin method, the weighted residual R is forced to be zero on an average sense over the problem domain. Linear independent weighting functions are chosen to be equal to the linear basis functions, and the approximating integral (equation 4) is formed.

$$\iint_D R \, \phi_i \, dx \, dy = 0 \qquad (4)$$

Where:

i = 1 to the number of nodal points and ϕ_i is the weight equal to the original basis function. Substituting the definition of R (equation 3) into equation (4) yields equation (5).

$$\iint L[\sum_{j=1}^{n} \hat{h}_j \phi_j) \, \phi_i dx \, dy = 0 \qquad (5)$$

After carrying out the integration in equation (5) on a piece-wise, element by element basis, incorporating the boundary conditions, a system of "n" equations and "n" unknowns is formed as shown in equation (6).

$$[A]\ \{\hat{h}\} + [B]\ \{\frac{d\hat{h}}{dt}\} + [D] + [E] + [F] = 0 \qquad (6)$$

where:

[A] is an n x n matrix representing the spatial variation in transmissivity.

[B] is an n x n matrix representing storage in the aquifer

[D] is an n-dimensional vector representing specified distributed recharge and discharge sources

[E] is an n-dimensional vector representing specified point sources and sinks.

[F] is an n-dimensional vector representing specified gradient boundary terms

Evaluation of the time derivative is performed utilizing a fully implicit Finite Difference approximation as shown in equation 7.

$$\frac{dh}{dt} = \frac{\hat{h}_{t + \Delta t} - \hat{h}_t}{\Delta t} \qquad (7)$$

When equation (7) is substituted into equation (6) the result is equation (8) which is solved by either banded Gauss elimination or Point-Iterative Successive Over-Relaxation.

$$([A] + \frac{1}{\Delta t}\ [B])\ \{\hat{h}_{t+\Delta t}\} = \frac{1}{\Delta t}\ [B]\ \{\hat{h}_t\} - [D] - [E] - [F] \qquad (8)$$

where:

h_t represents the known head at the previous time step

$h_{t + \Delta t}$ represents the unknown value to be solved for

[D], [E], and [F], known, specified prior to the current time step

[B] $\{h\}_t$ result from the previous time step

3.2 Model Data and Assumptions

A large body of information has been collected at the Arsenal in an attempt to define the geohydrologic system in relation to contaminant migration. This is particularly true for the area in Sections 23 and 24 from Basin "F" to the North Boundary. The primary source of data for this study was the "North Boundary Containment/Treatment System Performance Report" (NBCTS) which was compiled and written by Army personnel in 1985. Numerous additional data sources were utilized in development, calibration, and testing of the model. The majority of this data was collected during investigations performed for the purpose of siting, design, and evaluation of the barrier, system and as a result the most detailed data is for the area in the nearby vicinity of the barrier. This was quite fortunate in that detailed modeling results are desired for this part of the model. A complete listing of the data sources is found in the references.

3.2a Development of The Finite Element Mesh

The finite element mesh depicted in Figure 3.1 was developed to represent the alluvial aquifer in the nearby vicinity of the North Boundary Barrier System. The limits of the mesh were selected to allow simulation of operation of the barrier system and surrounding aquifer in sufficient detail to meet the goals of the study.

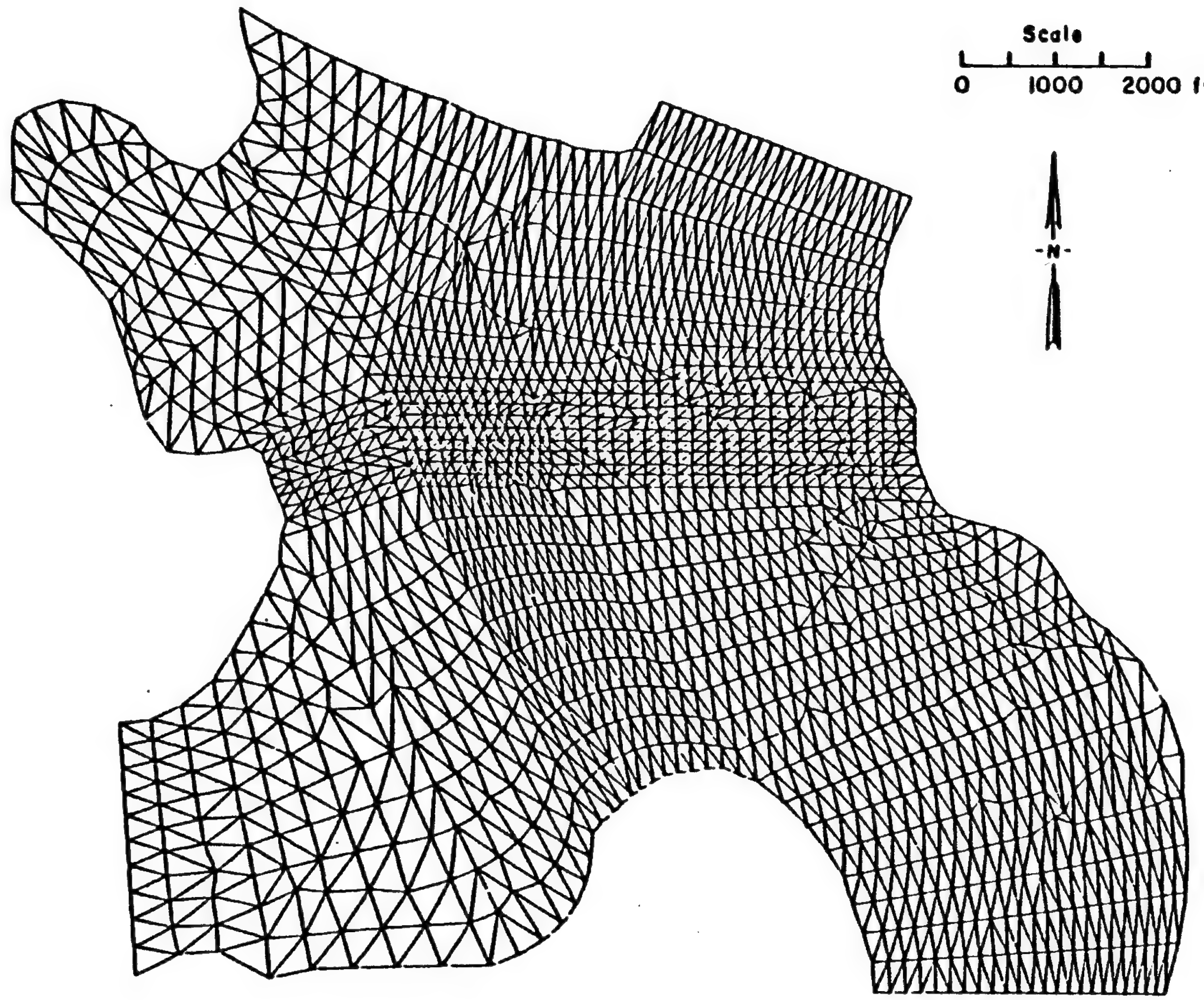


Figure 3.1 The Finite Element Mesh

In addition to mapping provided in the North Boundary Containment/Treatment System Report, observation well and test boring records were utilized to define the areal extent of the alluvial aquifer to be modeled and hence the limits of the finite element mesh.

A total of four separate finite element meshes were utilized during the progress of this study to complete calibration and verification of the north boundary barrier system from pre-barrier conditions, to modeling of the operation of the pilot and full barriers. The only substantive difference between three of the meshs was that lines of interior no-flow boundary nodes were utilized to represent the bentonitic slurry walls of the pilot and full barriers. During the study, the concept of recharge trenches of 45 feet down gradient of the bentonite slurry wall was proposed. To better simulate these recharge trenches, two additional lines of nodes were added to the model mesh immediately downgradient of the slurry wall. The final full-barrier mesh utilized to simulate the present and future operation of the barrier consists of nodal points, and elements. Each of thirty-five pumping, and thirty-eight recharge wells corresponded to a unique nodal point within the model (Figure 3.2). This allowed specification of individual pumping rates to simulate operation of the barrier system. Similarly, thirty-six monitoring wells were represented in the mesh to allow direct comparison of model results to field observations.

The total area represented by the mesh was approximately 2.78 square miles. Nodal spacing varied from approximately 125 feet in the vicinity of the barrier system to slightly in excess of 850 feet in the off-post area to the north of the Arsenal. Element size similarly ranged from approximately

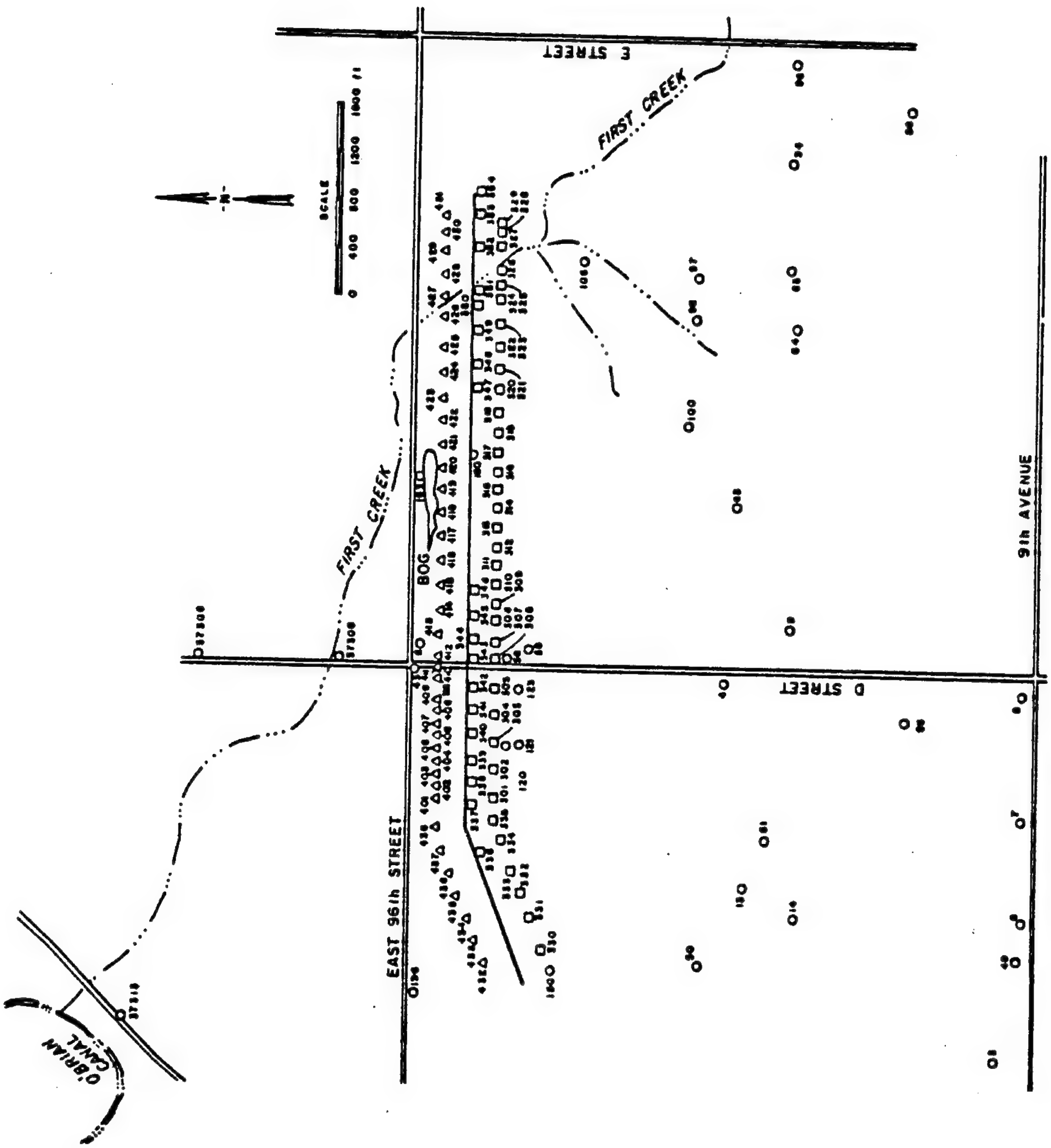


Figure 3.2 Present System Layout and Well Location Plan

Legend

Circles = monitoring wells, Squares = discharge wells
Triangles = Recharge Wells

0.14 to nearly 1.4 acre with an average of 0.6 acre. This density of discretization of the problem was required in order to provide detailed simulation results that are the goals of this study.

Hydrologic features such as First Creek, the bog and off-post First Creek impoundment are represented by nodes and elements, respectively. The significance of these features in the modeling of groundwater system in the vicinity of the north boundary is discussed in section 3.2.c to follow.

3.2.b Boundary Conditions

No-flow boundaries were utilized to simulate contacts between the Denver Formation and the Alluvium at the limits of the model where the contact is parallel to the predominant direction of groundwater flow. Interconnection of the alluvium and the Denver Formation has been documented, however because these boundaries were parallel to the predominant directions of flow and gradient,it is felt that the interchange of water between the formations is thought to be negligible.

Constant head boundaries were specified at the northwest (off-post) limits of the model corresponding to O'Brian Canal. Based upon water level monitoring data from off-post well 37313 adjacent to O'brian Canal, the groundwater levels in this vicinity are apparently controlled mainly by the water level in the canal. Because the water levels in the canal fluctuate with seasonal irrigation, the water table in the alluvial aquifer at this location shows considerable variation.

Constant head boundaries were specified at the northeast (off-post) boundary which represents outflow in the alluvial aquifer to the north east. Likewise, constant head and/or specified flux boundaries were utilized during phases of the calibration/verification at the southern limits of the

model area. These boundary conditions simulate underflow from up-gradient alluvial aquifer/Denver formation.

At the southern limit of the model, saturated thickness of the alluvium is generally less than ten feet. Previous investigations have indicated that saturated alluvium is in hydraulic contact with poorly cemented and indurated sand lenses of the Denver Formation.

At these locations, the permeability contrast between the alluvium and is relatively small and hydraulic gradients indicate that flow out of the Denver Formation into the alluvium is likely occurring. This contact is particularly suspect along the southern limit of the model, where sections 23 and 24 meet (Figure 2.3).

3.2.c Hydrologic Features

Surface hydrologic features within the model area include First Creek, tributaries and surrounding marsh, the north boundary bog, and the off-site First creek impoundment located approximately one-half mile north of the Arsenal.

First creek enters the model area at the east and southeast from sections 19 and 25 (Figure 3.2). Several surface hydrologic investigations have been performed on First Creek within the RMA. The The primary purpose of the studies were to evaluate flood risk and conveyance. Despite this and three previous groundwater model studies of the RMA North Boundary and vicinity, (Konikow (1977), Robson and Warner (1976 and 1977), little information exists on the interaction between First creek and the groundwater flow system. Comparison of groundwater levels in the vicinity of First creek with invert elevations (U.S.G.S MSL) of the stream give an

indication that First Creek and surrounding wetlands serve as a groundwater discharge zone in western section 19 and eastern section 24. The discharge zone begins roughly at the north-south midpoint of sections 19 and 24 and apparently ends at the confluence of First Creek and the RMA wastewater treatment plant discharge channel. The vegetation in this area is dominated by cattails and other wetlands species which are good indicators of the shallow groundwater levels. This area was modeled as a line of constant head nodes set at the approximate stream invert elevations based upon U.S.G.S topography. For the remainder of First Creek in the model area, no definite discharge/recharge relationship was apparent based upon groundwater and stream invert elevations. The same was true for the numerous drainage ditches and diversions located within sections 23 and 24. For better definition of the relationship of these features to the groundwater system more detailed surface topographic information would be required. Likewise for the off-site reaches of First Creek, there was insufficient detail of monitoring data to document such a relationship. Any stream-aquifer interaction in these areas was assumed to be negligible or unimportant to modeling the boundary barrier system operation.

The north boundary bog is a hydrologic feature of relative importance to modeling the north boundary of the RMA. It is commonly accepted that prior to installation of the barrier system, the bog served as a groundwater discharge zone. Since the discovery of hydrologic contamination at the north boundary, the bog has been bermed and contained to prevent surface migration of bog water. At present, treated groundwater is being discharged to the bog and allowed to recharge. The bog was modeled as distributed discharge (evapotranspiration) for the pre-barrier condition and net recharge under the current barrier operation.

The off-post First Creek impoundment was modeled as a series of constant head nodes that represent the average annual balance of groundwater inflow and outflow (recharge) from this impoundment. It is anticipated that this impoundment results in a net recharge of water from First Creek runoff to the subsurface.

3.2.d Potentiometric Surface

The model was calibrated to an interpreted February-March 1978 potentiometric surface shown on Figure 3.3. Because no comprehensive potentiometric data was available for the entire model area at this time, the potentiometric surface is a composite of information obtained from several sources. For the majority of sections 23 and 24 the water level information is from mapping performed as a part of the pilot barrier system design investigation (Battelle Columbus Labs/D'Appolonia Inc.,1979). For the vicinity of First creek in sections 19 and 24, and off-post areas, mapping performed by Konikow (1975), Romero and Ward, Ward and Sobol (1982), Zebel (1979),and Geraghty and Miller Inc. (1979) were utilized along with RMA monitoring well records to produce the map shown in Figure 3.3.

3.2.e Saturated Thickness

A map of saturated thickness was obtained by subtracting the base of alluvial aquifer data (top of Denver Formation) from the February-March 1978 potentiometric water surface elevation. The resulting map is shown in Figure 3.4. Saturated thickness ranges from zero at the limits of saturated alluvium to in excess of 35 feet in the deeply incised paleochannels running from Basin "F" to the north boundary. Average saturated thickness is indicated to be a little over 14 feet. Base of alluvial aquifer elevation

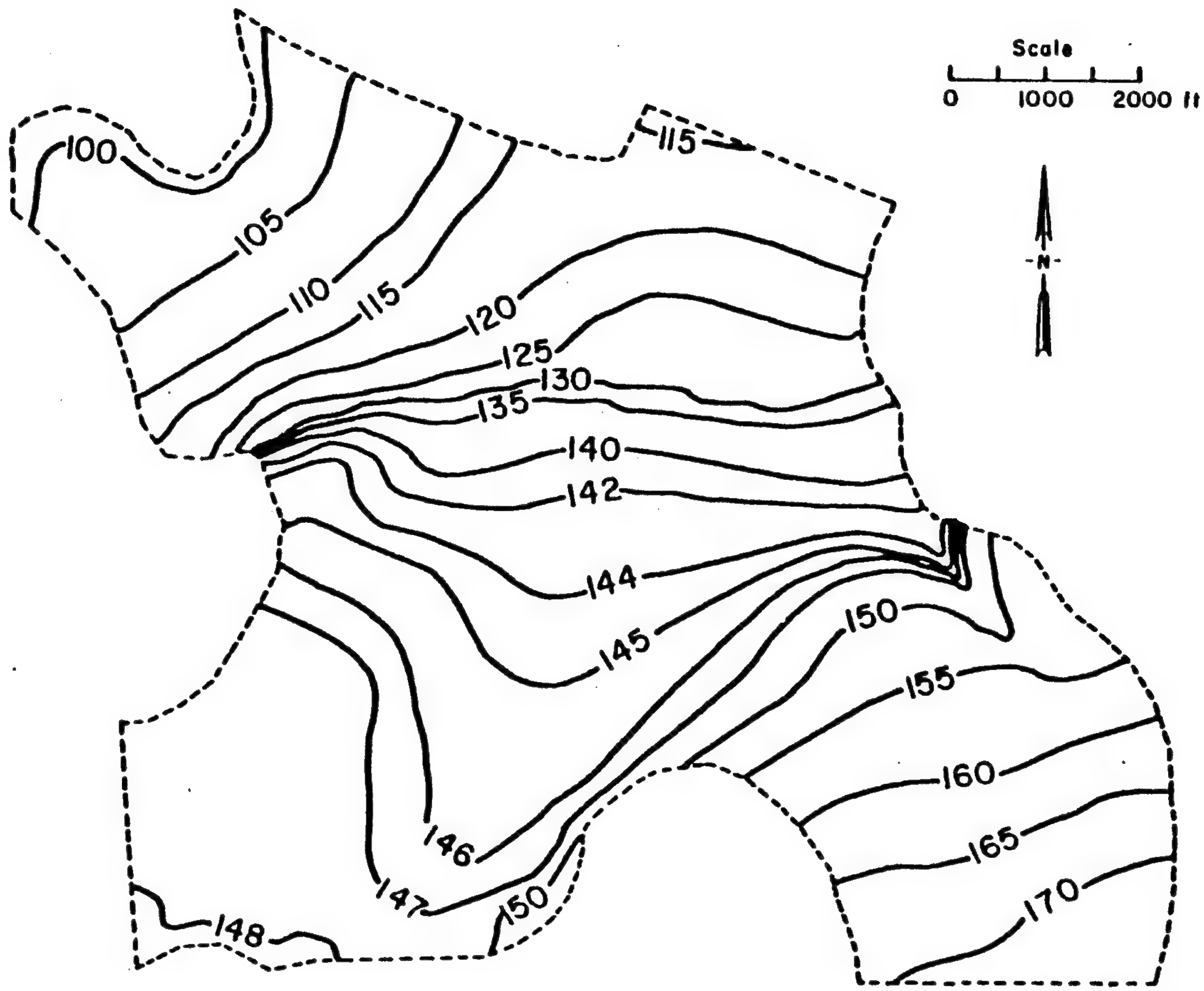


Figure 3.3 February-March 1978 Pre-system Potentiometric Surface
(contours in feet, add 5,000 for AMSL)
Contour Interval Varies

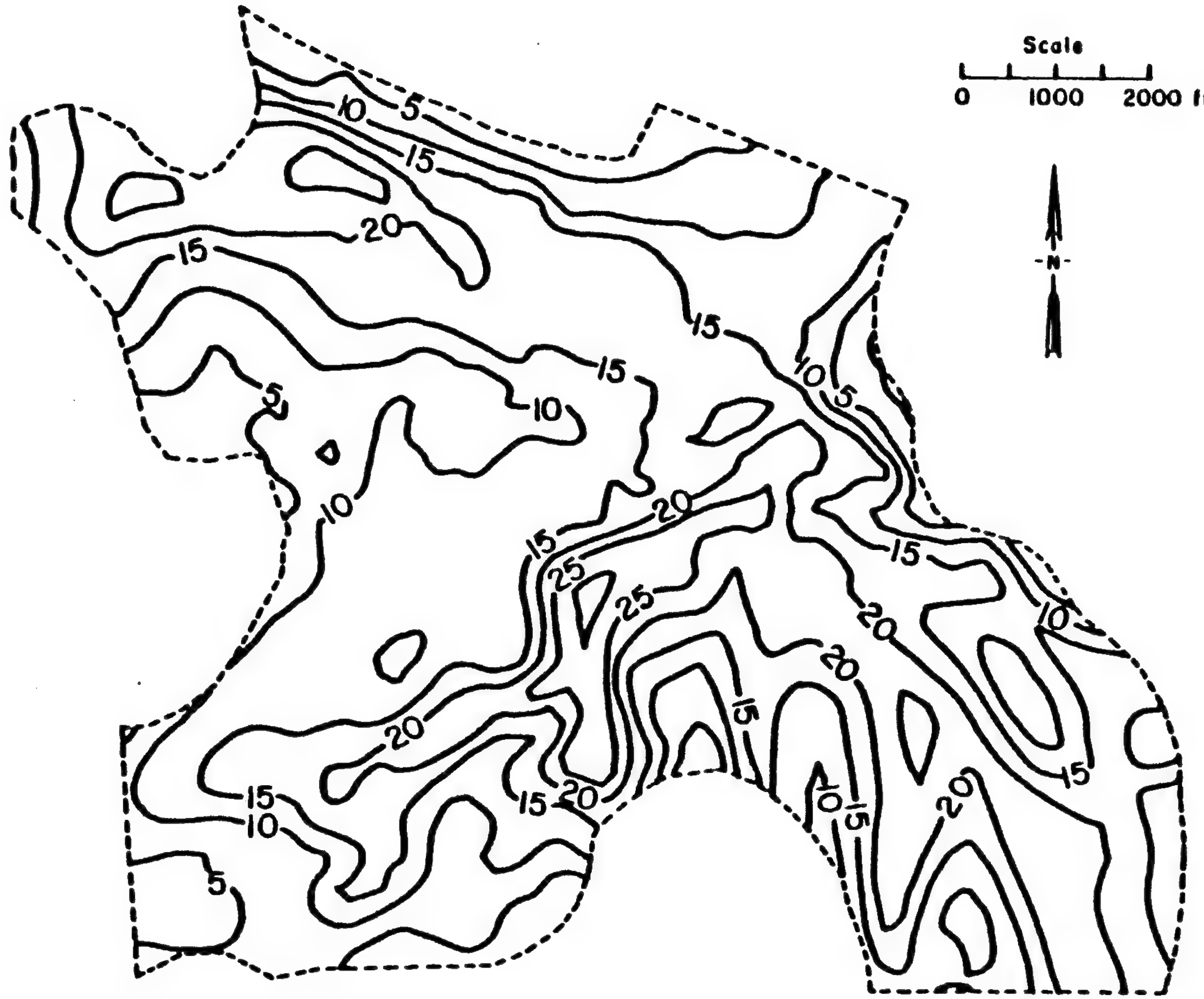


Figure 3.4 Pre-system Saturated Thickness Map
(contours in feet, interval varies)

data was obtained from mapping provided in the "North Boundary Containment/Treatment System Performance Report", RMA, 1983.

3.2.f Hydraulic Conductivity and Transmissivity

Several aquifer tests were performed within the model area as a part of previous hydrologic, or north barrier/pilot system design investigations. Five, 72-hour pump tests were performed by the U.S. Army Corps of Engineers, Waterways Experiment Station (WES) in 1978 (Visipi). The results are shown on Table 3.1. For location of the test sites, please refer to Visipi,(1978). Several shorter-term aquifer tests were performed in the vicinity of the pilot barrier system. The precise location of these

Table 3.1 Aquifer Test Results for the Model Area

Test Designation	Average Transmissivity	Source
A	4,400 ft^2/day	Zebel, 1979
B	7,500 ft^2/day	Zebel, 1979
C	11,000 ft^2/day	Zebel, 1979
D	1,600 ft^2/day	Zebel, 1979
E	360 ft^2/day	Zebel, 1979

tests were not apparent, but the results of this testing are in general agreement with that of WES testing in the vicinity of the north boundary.

In addition to the pumping tests, numerous slug-tests were reported for the model area by the U.S. Army Waterway Experiment Station (WES) Zebel (1979). The results of the slug test do not always agree with that obtained from long-term aquifer testing. The slug tests generally indicate lower hydraulic conductivity values. The reason for this discrepancy is primarily due to the fact that slug tests sample a very small part of the aquifer and are often effected by well bore storage and screen losses. The long term

pump test results probably more accurately represent the average properties of the alluvial aquifer.

Model calibration to the February-March 1978 potentiometric surface was performed by varying hydraulic conductivity relative to these known points until a good agreement between model predicted and observed head was attained. The resulting model calibrated transmissivity map is shown in Figure 3.5 (transmisivity is the product of hydraulic conductivity and saturated thickness).

Hydraulic conductivity of the aquifer ranged over several orders of magnitude from a maximum of 3,000 ft./day (1.06cm/sec) in the deep gravel filled alluvial channels to a minimum allow of 0.01 ft./day (3.5×10^{-6}cm/sec)in the area of steep hydraulic gradient at the west end of the barrier system. As indicated on Figure 3.5 transmissivity varied from 30,000 to 0.1 ft^2/day. These results are in general agreement with aquifer, laboratory, and slug test performed in this area (Zebel,1979) and (WES, 1980). The results are in relative agreement with previous modeling efforts by (Konikow, 1977), (Robson and Warner, Warner 1979).

3.2.g Storage Coefficient/Specific Yield

Previous model studies performed by the U.S. Geological Survey utilized specific yield values of 0.25 which would be representative of water yield due to gravity drainage for the coarse grained alluvial aquifer material present at the Arsenal (Robson and Warner 1979). This value was originally obtained from a regional water resource investigation along the South Platte River (Smith, et al., 1964). Where this value may be

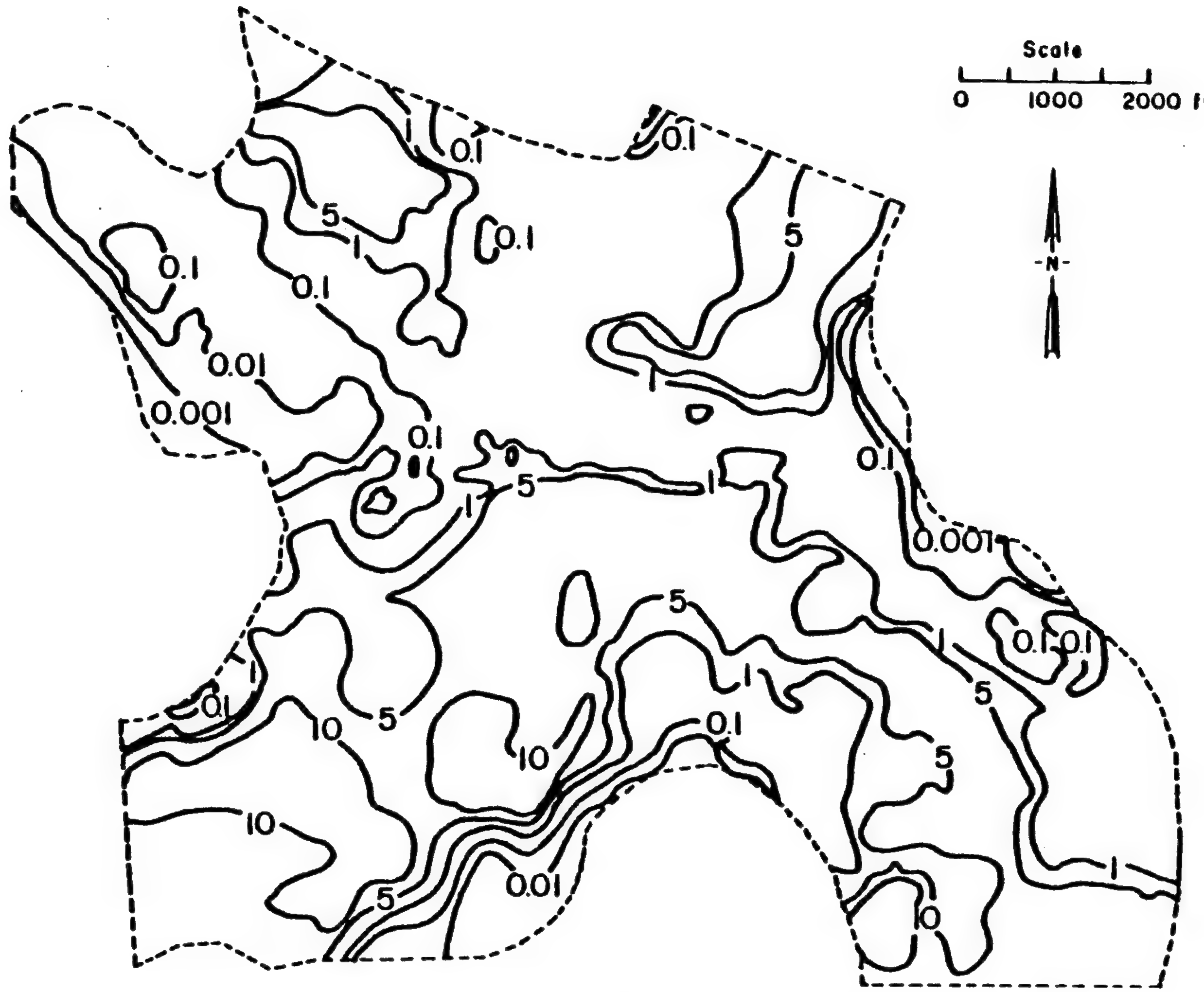


Figure 3.5 Model Calibrated Transmissivity Distribution

(units of thousand ft^2/day, contour interval varies)

representative of the regional aquifer and other parts of the RMA, more recent aquifer testing and mapping in Sections 23 and 24 indicate that the alluvium is confined, or partially confined by overlying fine grained aeolian deposits. Review of cross-section and water level information over the recorded history of this part of the RMA indicates that this is true for the recorded range of potentiometric history including recent operation of the barrier system. Results of long term (72hr) pump testing indicate a range of storage coefficient values from 0.25 down to 0.001 with an average of about 0.05. This average value was utilized for transient calibration/verification and subsequent simulations. This value is thought to adequately represent the average water yielding characteristic of the alluvial aquifer.

3.3 Model Calibration and Verification

Model calibration consists of refinement of the model input data from the initial estimates to reach a good fit between observed and model calculated results. A Unique solution of a groundwater flow problem requires information including: aquifer parameters such as transmissivity and storage coefficient initial head conditions, boundary conditions, and location and magnitude of applied stresses such as pumping, recharge and evapotranspiration. A unique solution is attained only when the proper combination of the above parameters are selected such that the actual physical problem is accurately represented.

The calibration procedure typically begins with selective elimination or definition the parameters listed above. Parameters which are known or can be reasonably estimated, are specified in the input data. Parameters that

may be known or estimated only at discrete points (eg. transmissivity and storage coefficient) are interactively refined and until their spatial variability are approximated. Calibration of a model should ideally include both steady and unsteady simulations.

3.3a Steady State Calibration

Calibration of the model began with refinement of model data to approximate the Feb-March 1978 observed pre-barrier system potentiometric surface. The 1978 water surface configuration was approximated as a steady state profile with no changes in storage occurring. This assumed that aquifer stresses and boundary conditions were in dynamic equilibrium during this period. Although a natural system is never truly steady state, the thought is that the rate of change of the aquifer stresses and boundary conditions are quite small under natural conditions with no great short-term variation in recharge or discharge.

Steady-state calibration was performed primarily by varying aquifer hydraulic conductivity in an iterative fashion until the desired fit was obtained. Results of the steady-state calibration are listed in Tables 3.2 and 3.3.

Table 3.2 provides a statistical measure of the goodness of fit to the observed potentiometric surface. Table 3.3 provides a comparison of model computed heads versus observed monitoring well information. Unfortunately, only a few of the 36 monitoring wells represented in the mesh have observed data recorded for this period. A map of calibration error is provided to allow review of spatial distribution of the error over the model area (Figure 3.6). Figure 3.7 depicts the model computed pre-barrier potentiometric surface.

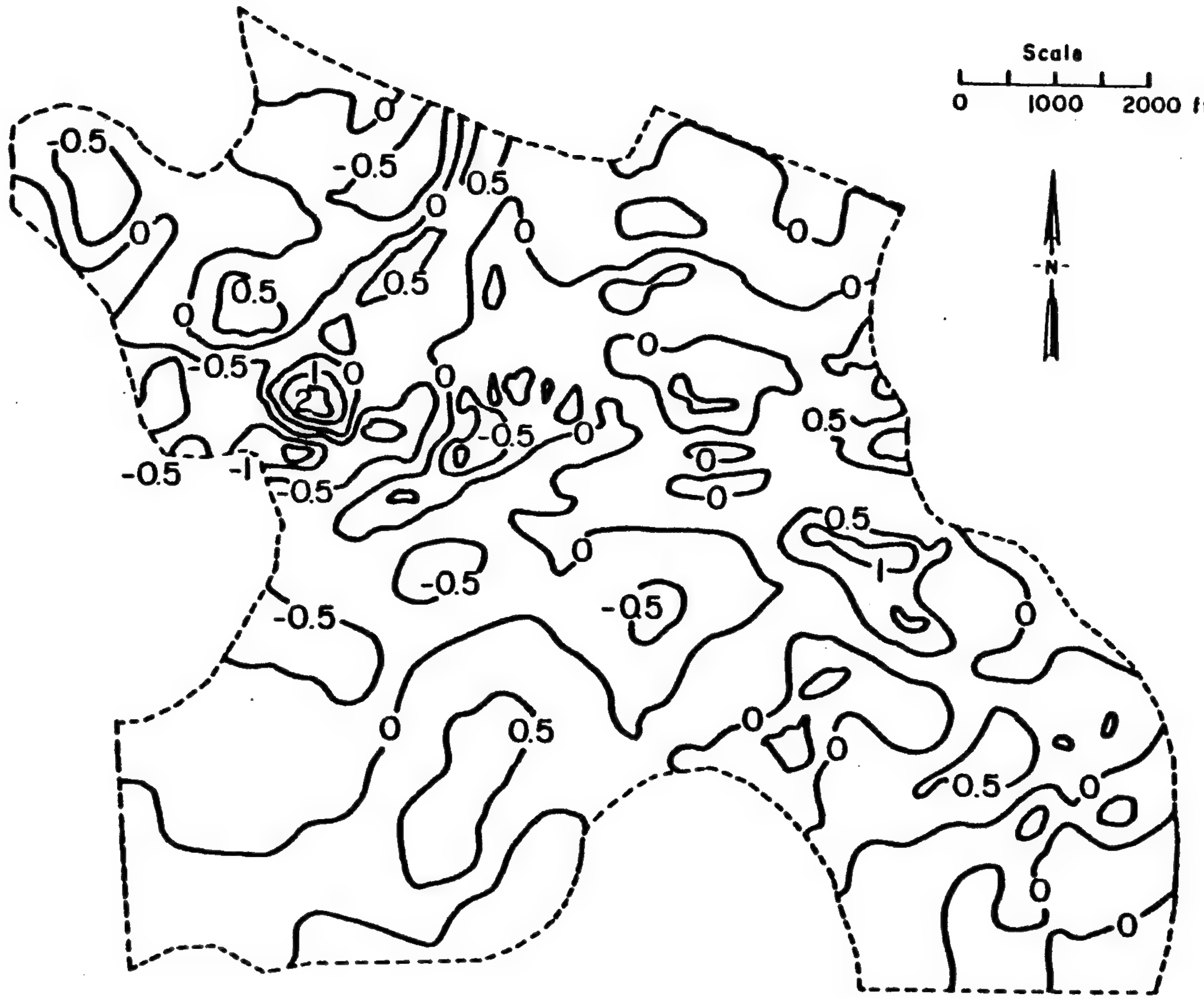


Figure 3.6 Feb-March 1978 Observed versus Model Calibrated Head Differences (contours in feet, interval varies)

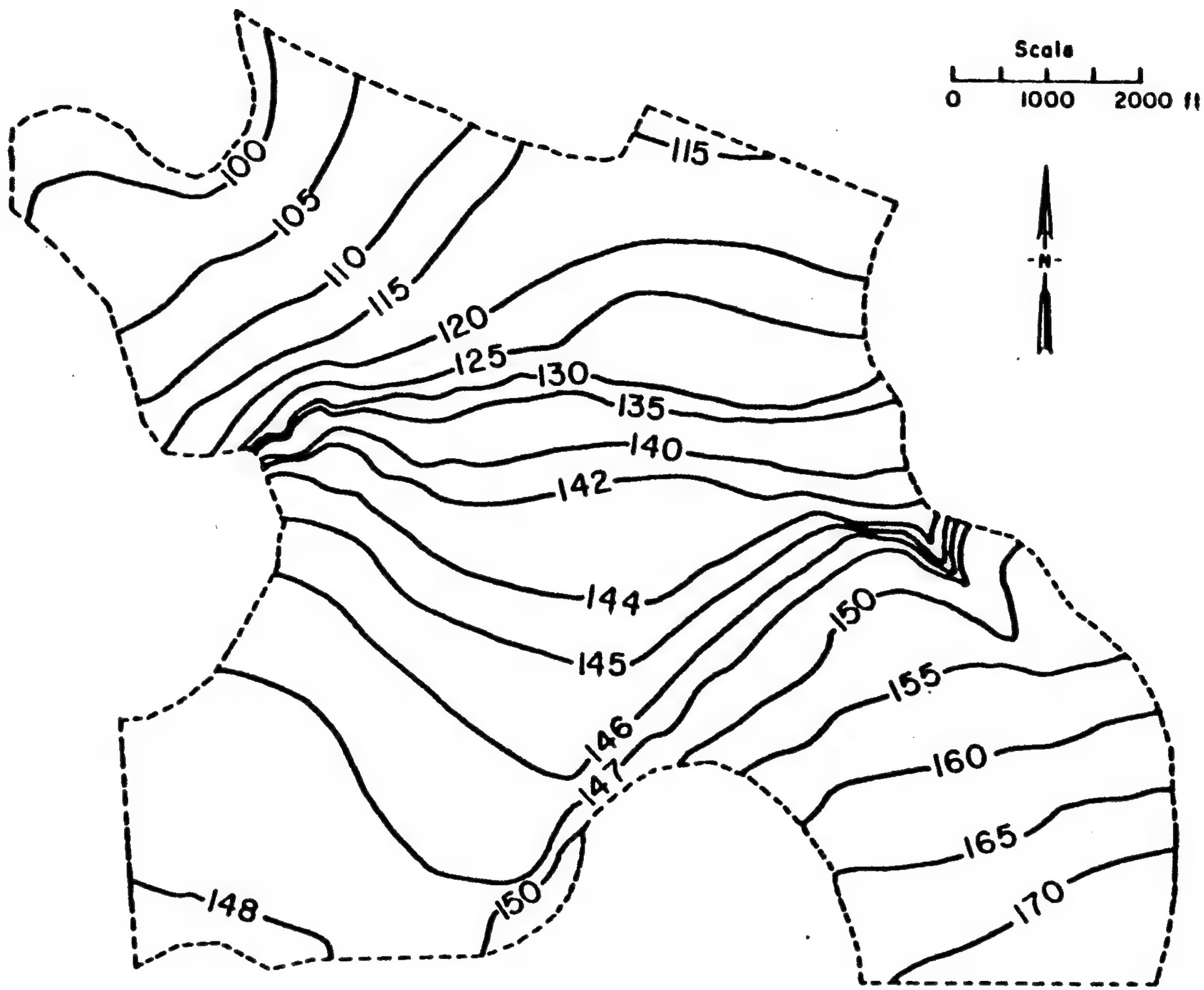


Figure 3.7 Model Computed Potentiometric Surface February-March 1978
(contours in feet, add 5,000 for AMSL, contour interval varies)

Table 3.2 Statistical Comparison of Model Calculated Vs. Observed Potentiometric Surface Data, February-March 1978

Head Difference	Percent Node Within:(1)	RMA Criterion
± 0.25 ft.	70.65 %	None Specified
± 0.50 ft.	70.72 %	None Specified
± 1.0 ft.	92.85 %	65%
± 1.5 ft.	96.98 %	80%
± 2.0 ft.	98.90 %	90%
± 5.0 ft.	100.00 %	None Specified
Area-weighted mean squared error	0.296feet	0.4feet

Notes:
(1). Does not include constant head nodes.
(2). Average Absolute Error at a node -0.025 feet.
(3). Area-weighted absolute average error = -0.00173 feet.

Review of Tables 3.2 and 3.3 indicate excellent agreement between model calculated and observed data. On the average the model is accurate to within ± 0.3 feet. At nearly 99 percent of the nodes the model calculated and observed data are within ± 2.0 feet, and at almost 93 percent of the nodes the difference is less than 1 foot.

The greatest total error in the model occurs in the area at the extreme western limit where the hydraulic gradient is very steep and poorly defined. In this area, the saturated thickness is very small and the hydraulic conductivity is in the range for the fine grained clay-shale of the Denver Formation. Estimates are that the error in this area accounts for less than 1.5gpm of total flow for the barrier and is not considered to be a significant problem for simulation of operation of the barrier system.

Figure 3.8 depicts the model calculated equilibrium flux components of the water balance for February-March 1978 potentiometric conditions. Based

Table 3.3 Observed vs Model Calculated Heads
February-March 1978, Pre-Barrier Calibration

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	13	**		
23006	25	**		
23007	48	**		
23049	17	**		
23096	102	**		
Central Section 23				
23004	223	5145.75	5146.0	-0.25
23013	98	5148.	5147.47	0.53
23014	67	**		
23050	120	5148.	5147.49	0.51
23051	82	5146.6	5147.19	-0.59
West-Central Section 24				
24009	230	5146.	5146.5	-0.5
24053	491	**		
East-Southeast Section 24				
24064	867	**		
24065	1003	**		
24088	1200	**		
24094	1215	**		
24095	1379	**		
24097	1079	**		
24098	981	**		
24100	782	**		
24105	1274	**		
Barrier Wells-Upgradient				
23120	323	5141.	5141.3	-0.3
23121	316	**		
23123	455	5142.	5142.02	-0.02
23150	166	5146.	5145.99	0.01
24023	579	**		
24056	558	**		
24058	553	**		
24180	971	**		
Barrier Wells-Downgradient				
23043	590	5135.5	5136.24	-0.74
23196	379	**		
24006	640	5136.5	5136.6	-0.1
24163	1021	**		
Off Post-North				
37308	740	**		
37309	805	**		
37313	681	**		

(1). Node Number - Pre-Barrier Calibration Mesh
(2). Observed heads interpolated heads from:"Potentiometric Surface Map - Pre-System, Evaluation North Boundary Pilot Containment System, RMA, Denver, Colorado", Battelle Columbus, 1979.
** No observed data recorded.

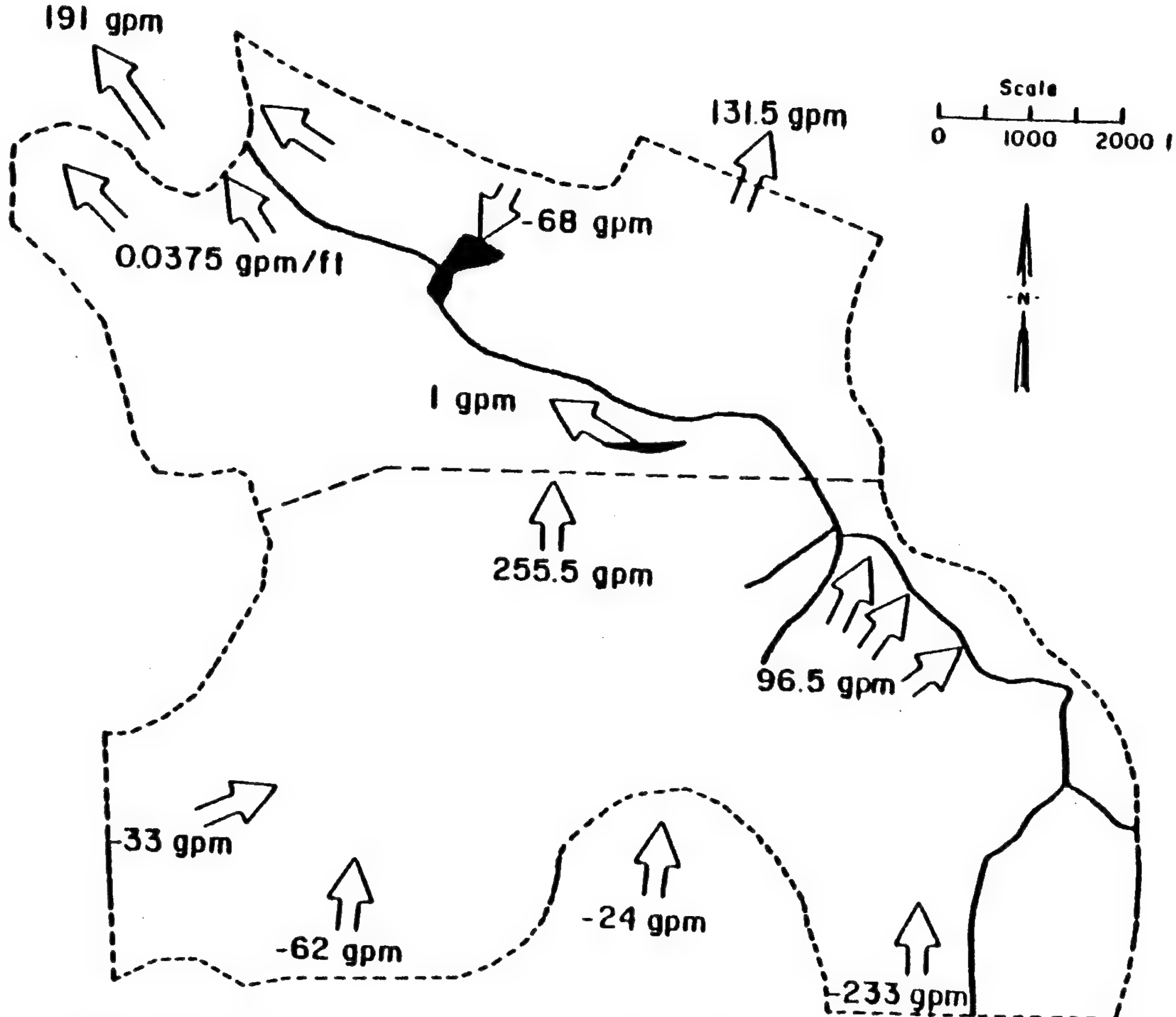


Figure 3.8 Model Computed Equilibrium Flux Components of Water Balance
February-March 1978 Potentiometric Surface

upon the model results, approximately 255 gallons per minute (gpm) crossed crossed the RMA North Boundary in the alluvial aquifer under these conditions. Discharge to First Creek and surrounding marshes was approximately 96gpm (0.21 cfs), or 0.044gpm per lineal foot of stream. This result is within the range of recorded low (non storm) flow of First Creek (0.0-0.3 cfs) (Resource Consultants 1982). A net recharge of 68gpm (0.15cfs) from the off-post First Creek impoundment is indicated. Utilizing the estimated 220,000 square-foot of surface area of this impoundment, the leakage rate would result in a decline in pond water level of less than three quarters of one inch per day. In actuality, inflow to this pond from First creek could easily support this model calculated leakage rate.

3.3.b Transient Calibration and Verification Data Sources and Uncertainty

Subsequent to completion of steady-state pre-barrier calibration, transient calibration/verification was performed to test the model under time varying conditions and further define and refine the predictive capabilities of the model. From the February-March 1978 pre-barrier conditions to the present time, a total of about 10 years have elapsed. During this period, the present barrier system evolved from a pilot system that was installed in mid-1978, to the full barrier system as it presently is configured. The start-up of the pilot system reportedly occurred on July 21, 1978.

The pilot barrier system operated for approximately three, and one quarter years. During this time the need for a full, complete cut-off was identified due to evolving water quality restrictions and aquisition of

additional water quality data. The full barrier system was fully installed and operational during the period of November 1981 and January 1982.

Transient calibration and verification involved simulating the aquifer response through time due to operation of the pilot and full barriers systems. The capability to re-create the history of barrier operations and aquifer response was found to be somewhat limited by the lack of continuous, detailed, pumping data for the barrier systems. The limitations and uncertainty of this data is discussed in greater detail in the section to follow. Despite the limiting data, utilization of what data was available yielded quite favorable results in terms of comparison of observed and model computed water levels.

Figure 3.9 presents the time relationship between available barrier operation data, and monitoring well records. At regular intervals over the 10 year period, comparison of model computed to observed data is presented for up to thirty-six monitoring wells. These intervals are also indicated on Figure 3.9

Approximately three and one-third years of barrier operation data was available for the nine year period from startup of the pilot system to the present day. The 40 months of data include eight months of pilot system operation, and thirty-two months of full barrier operation information.

A total of eight months of weekly average pumping and recharge rates were recorded for each well in the pilot barrier system from startup July 21, 1978 to March 14, 1979. This data was presented in three reports entitled "Pilot Containment System Monthly Updates" (RMA, USATHAMA, 1979) and represents the most detailed operations data available for the entire period of record to the present day. Sources of potentiometric information for this same period include: "Evaluation of North Boundary Pilot

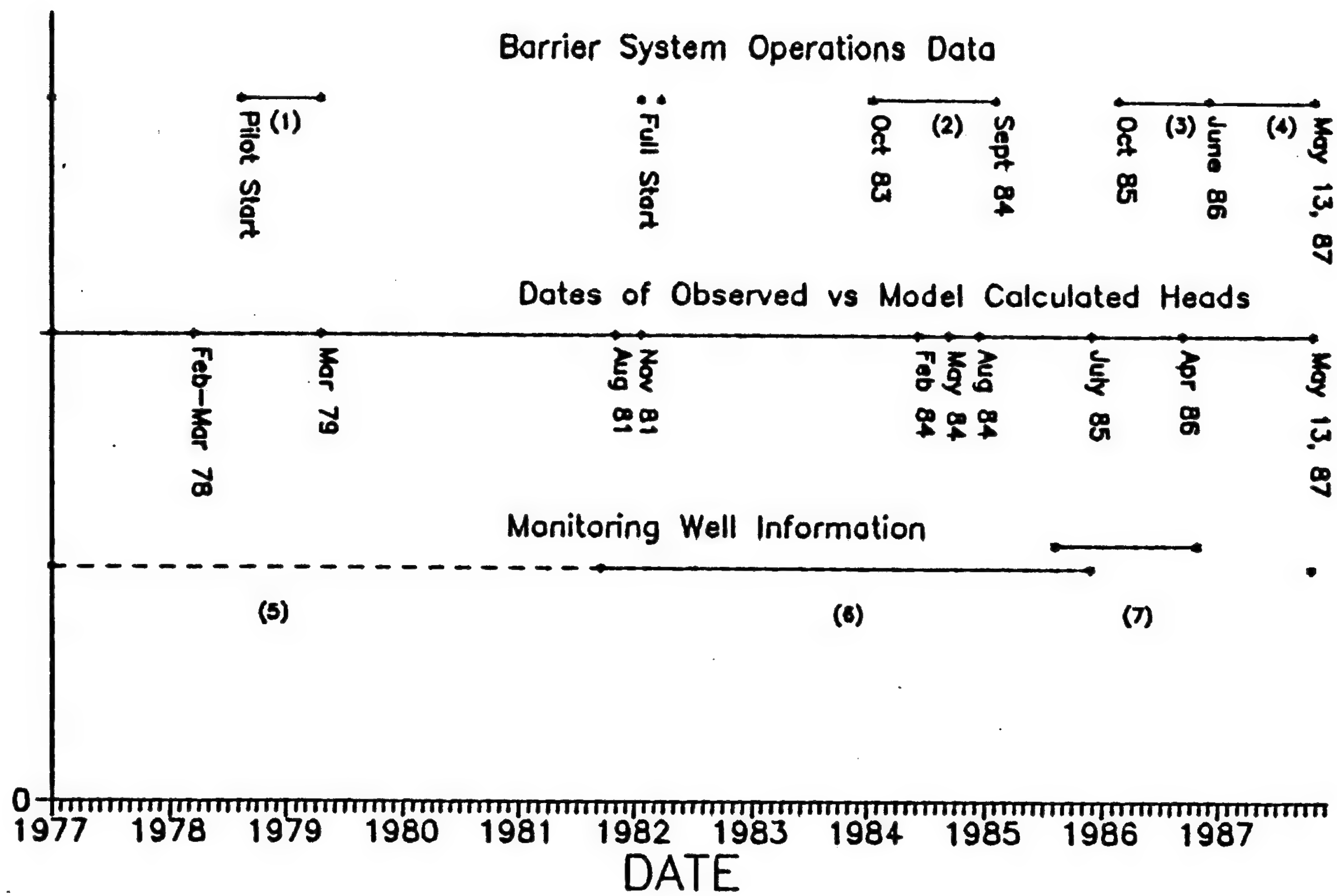


Figure 3.9 Time Relationship Between Barrier Operations and Monitoring Well Data and Model Predicted Results
(see next page for notes 1 through 7)

Data Sources Utilized- Figure 3.9:

1. Several Data Sources Including:
   "Evaluation of North Boundary Pilot Containment System, RMA",D'Appolonia Inc, Battelle Columbus Laboratories, Columbus, Ohio, 1979.
   "Pilot System Monthly Updates" RMA, 1981.
2. "North Boundary Containment/Treatment System Performance Report", RMA, USAE WES, 1985.
3. "Rocky Mountain Arsenal North Boundary Flow Rates: October 1985-June 1986., D.P. Associates, Inc., 1986.
4. Uncompiled barrier operations records, Greg Ward, Rudy Sweder, RMA.
5. Several Data Sources Including:
   "Basin "F" to the North Boundary, Rocky Mountain Arsenal, Volume I, Geotechnical Definition, Zebel, 1979.
   "Evaluation of North Boundary Pilot Containment System, RMA",D'Appolonia Inc, Battelle Columbus Laboratories, Columbus, Ohio, 1979.
   "Pilot System Monthly Updates" RMA, 1981.
6. "RMA Water Level Statistics and Plots, Volumes I and II, D.P. Associates, Inc., 1985.
7. "Water Elevation Report for the North Boundary System: April 1985 through May 1986",D.P. Assoc. Inc, 1986.

Containment System, RMA, Denver CO" (Battelle Columbus Labs,1979), and "Basin "F" to North Boundary, Vol 1. Geotechnical Definition, Zebel, 1979). The majority of this data are for wells in the immediate vicinity of the pilot containment system. No data was available for central and east-southeast sections 24, or the off-post portions of the model.

A full twelve months of barrier operation data (FY 84) was contained in the "North Boundary Containment/Treatment System Performance Report" (NBCTS) (Figure 3.9). This information was limited to weekly average adsorber flow-through data for each of the three adsorber and manifold systems. No information was presented as to the distribution of the manifold flows by discharge or recharge well. For this same period, potentiometric levels were available for the majority of monitoring wells in the model area. Potentiometric levels were recorded on a quarterly basis starting in July of 1981 until July of 1985. The number and distribution of monitoring wells have continually increased with time.

An additional twenty months of full barrier operation data was recorded for fiscal year 1986 to May 1987. This data consists of weekly and monthly average pumping and recharge rates for each individual well in the system as well as monthly, weekly, and daily total adsorber/manifold flow-through rates. This data was contained in a report entitled "Rocky Mountain Arsenal North Boundary Flow Rates, October 1985 - June 1986",July 1986, D.P. Assoc, Inc, and monthly operations records. Seemingly, the individual discharge and recharge well data would be very useful for transient calibration. However, inspection of the individual well data obtained in this report indicates that is extremely unreliable. On the average, the sum of discharge well pumping volumes should be equal to the adsorber flow-through and the sum of recharge. The data presented for October 85 through June 86 indicate that

these values do not ever agree. Discharge well and recharge well flows as presented, differed from adsorber flow-through rates by as much as 85 percent and as low as 3.6 percent during this period of record.

Discussions with Arsenal personnel indicate that the adsorber flow-throughs are thought to be most representative of system operation. The totalizing flow meters utilized for each pumping and recharge well are apparently subject to mechanical and electrical difficulties. Monitoring well information for the 1986 to present day period is incomplete. Regular quarterly collection of potentiometric data over the whole model area was discontinued in July of 1985. RMA personnel continued to obtain water level measurements in wells immediately adjacent to the barrier system. This data was summarized in a report entitled "Water Elevation Report for the North Boundary System: April 1985 through May 1986" (D.P. Assoc. Inc, 1986). A total of six monitoring wells represented in the finite element mesh are included in this data.

Geographic Distribution of Data Uncertainty

The ability to simulate the response of a natural hydrologic system is highly dependant upon the detail to which the system can be described in the calibration process. Characterization of groundwater flow systems always involves a degree of uncertainty. Boring logs, aquifer tests and other forms of exploration sample a very small part of the total area of interest. Hydrologists rely on interpretation based upon conceptual models of how the available data integrate into a picture of the the physical system. The degree to which this "picture" is based upon interpretation is inversely proportional to the density of data. In the case of a groundwater flow

system that is dominated by narrow, sinuous paleochannels such as is found at the RMA north boundary, the detail of information is particularly important. It was fortunate that the most detailed aquifer data is available in the vicinity of the barrier system where highly detailed results are desired. For other parts of the model area, the level of resolution of avaliable aquifer data is more limited. Areas of the model can be categorized according to the relative level of uncertainty that is associated with the detail to which the the hydrologic system can be described. On this basis, for the tables of comparison of observed versus model calculated potentiometric data, the observation wells are grouped according to geographic regions within the model area (Figure 3.10). The degree of uncertainty and limitations to absolute matching of observed data vary for each region.

Region 1-Barrier Wells, Upgradient and Downgradient

For areas within a short distance (< 250 feet) of the slurry wall barrier, the model solution is most limited by the description of pumping and recharge rates. Monitoring wells within a short distance of the barrier respond very rapidly to changes in pumping rates and show fluctuations on the order of 3-4 feet from quarter to quarter. The ability to match these potentiometric levels on a specific date is dependant upon short-term pumping and recharge data.

Region 2-South and Central Sections 23 and West-Central Section 24

For on-post areas within the limits of mapping provided in the NBCTS report and out of the immediate short-term influence of barrier operation,

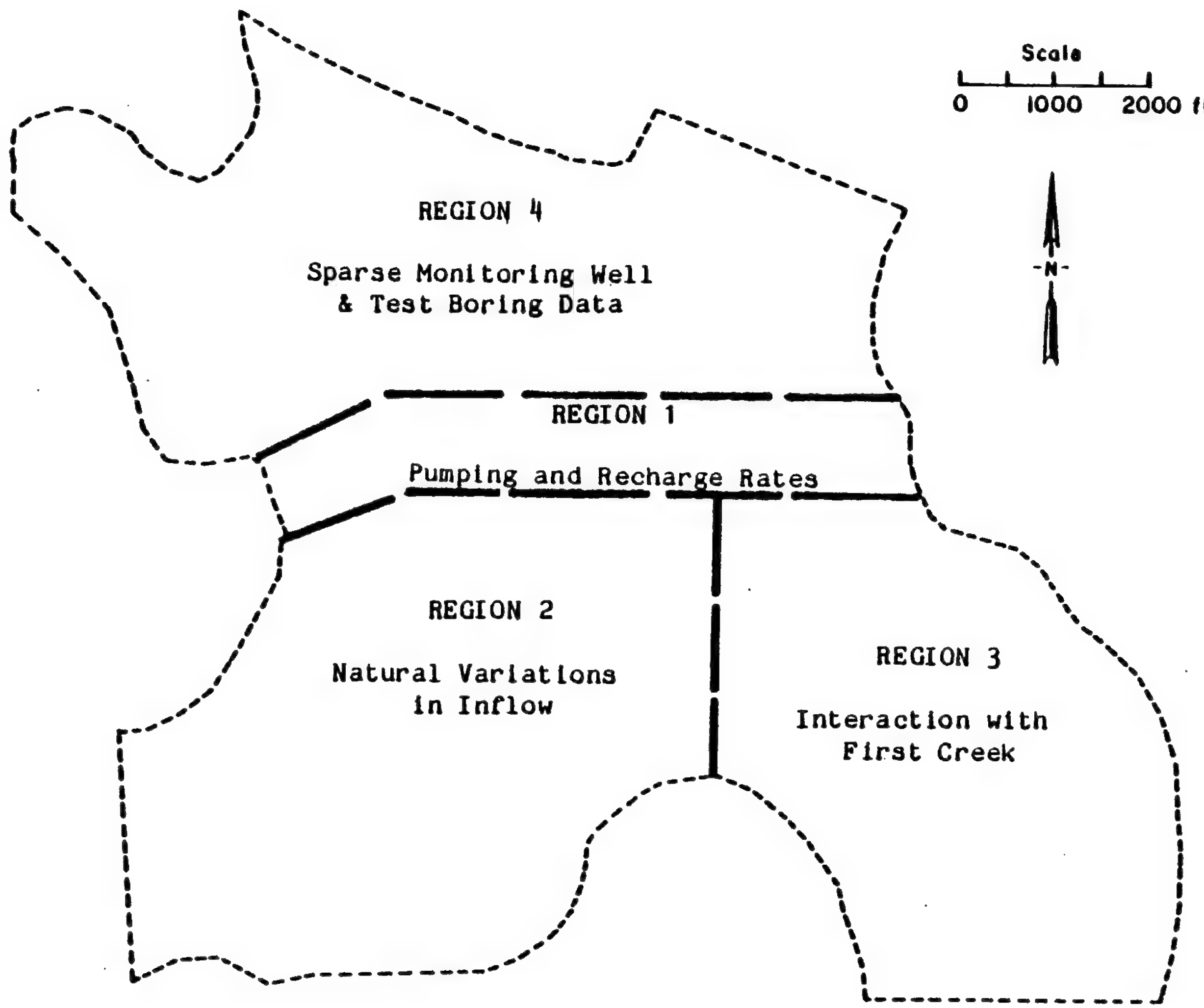


Figure 3.10 Regions of Data Uncertainty

(south and central sections 23, and west-central section 24), the effects of short-term barrier operation fluctuations are buffered by distance.

The observation points show much more stable well hydrographs with fluctuations due to seasonal and long-term variations in flow through the model area of generally less than two feet. The primary limitation to reproducing the observed data are limitations in description of seasonal variations in inflow to the model area.

Region 3-East-Southeast Section 24

For areas adjacent to First creek and tributaries,(East-Southeast Section 24), the primary limitations to modeling are the capability to describe the interaction of the groundwater system and First Creek and the limited availability of long-term (pre-barrier) monitoring information. Several of the monitoring wells in this area are located within very short distances of the creek, or drainage channels leading to the creek (24150, 24088, 24095, etc). Groundwater levels at these locations may be controlled by discharge, or recharge to and from the surface features. Since detailed survey data was not available to doccument stream water level and invert elevations in relation to groundwater levels, the ability to model these areas is somewhat reduced.

Region 4-Off-Post, North

The degree of uncertainty associated with the off-post area of the model is large relative to that in the immediate vicinity of the barrier system. In this area, very little test boring data is available. Definition of the aquifer thickness and initial potentiometric levels were based upon

the few monitoring wells which were included in regular quarterly monitoring efforts. Fortunately, the need for extreme detail in this area is minimal.

3.3.c Results of Transient Calibration and Verification

The results of transient calibration and verification are summarized in nine tables which provide direct, verifiable, comparison of model computed results and observed monitoring well records collected and compiled by the Arsenal. These tables present some 203 individual comparisons for approximately 9 years of record, for the 36 monitoring wells represented in the finite element mesh.

Comparisons were tabulated at regular intervals that were selected based upon availability of concurrent barrier operations and monitoring well records. Observation well records for a given month typically consist of depth to water measurements taken over a period of one to several weeks. Comparisons provided in this report for a given time, consist of all wells of record for the period, minus any records that appeared anomalous as compared to the history of that well. For monitoring wells that showed considerable variation from month to month, and model results indicated to be quite sensitive to barrier operations, the immediate range of potentiometric data is provided for comparison.

For the November 1981 summary, very little actual monitoring well data was available. This date represents the transition period between the pilot and full barriers. To provide documentation of model comparison at this critical transition period, the arithmetic average of October and December 1981 data was provided for comparison.

Potentiometric records for monitoring well 23049 were adjusted to reflect a top of casing elevation estimated based upon field observation.

The reported casing stick-up differed from that in the field by approximately 2 feet. Potentiometric levels reported for well 23049 typically were 2 feet higher than other wells in the same vicinity. In some cases potentiometric records for a well were augmented by records for wells immediately adjacent in the field. Such is the case for wells 23096 and 24053, where wells 23097 and 24049 were utilized.

Pilot System: (March 1978 to November 1981)

Transient calibration and verification began with running the model through time from the February-March 1978 calibration through July 1978 start-up of the pilot barrier system to installation and start-up of the full barrier system. In reality, transition from the pilot barrier to full barrier systems occurred over a period of several months. Information on the specific details of this transition period is not available. For purposes of this model study, it is assumed that the transition was instantaneous, nominally in November of 1981. Discussions with Arsenal personnel indicate that this is a good estimate for start-up of the full barrier. As previously indicated, individual weekly-average discharge and recharge rates were recorded for each of 18 wells in the pilot system for 8 months after start-up. The quality of this data is somewhat suspect in that the sum of discharge pumping does not always equal recharge, but it is the best information available for this period. The pilot system pumped at an average rate of approximately 50 gpm relying mainly on wells 331, 332, and 333 (full barrier wells 410, 411, and 412), for recharge after the first few months of operation. For comparison, steady-state computations utilizing pre-barrier potentiometric levels indicated that the pilot system naturally intercepted approx -imately 60-65gpm during this period. Barrier operation was assumed to remain constant according to the March 1979 recorded pattern for the

remainder of pilot barrier operations, when no detailed operations data was available.

Tables 3.4, 3.5, and 3.6 summarize 52 individual comparisons of model observed vs model calculated potentiometric levels for March 1979, August 1981, and November 1981. Over this interval of time, quarterly water level monitoring began, and consequently the number and distribution of monitoring well data increased dramatically.

Transient Calibration: March 1979

A review of Table 3.4 indicates that agreement between model calculated and observed potentiometric levels is quite good. The model appears to be accurate to within ± 1 ft at all wells. The greatest error being immediately down-gradient of the slurry wall at monitoring well 24006. This location is extremely sensitive to recharge rates of wells 331, 332 and 333, which are approximately 150ft. immediately up-gradient. Small fluctuations in recharge rate (<2gpm) result in responses in well 24006 on the order of several tenths of a foot.

Transient Calibration: August 1981

For August 1981, (Table 3.5) observed data is recorded for a total of 26 monitoring wells. In general, the agreement between these data and model computed is excellent, within ± 1.0ft. The exception being for wells 24006 and 23123 immediately in the vicinity of the pilot barrier and wells within East-Southeast Section 24 and off-post areas. For wells 24006 and 23123, considering that no pumping data was available for this period, and the sensitivity of these wells to pump rates, the comparison is fairly good.

Table 3.4 Observed vs. Model Calculated Heads-March 1979

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	**		
23006	25	**		
23007	48	5145.9(a)	5146.23	-0.33
23049	17	5146.7(a)	5146.49	0.21
23096	102	**		
Central Section 23				
23004	223	**		
23013	98	**		
23014	67	**		
23050	120	**		
23051	82	5144.8(a)	5145.59	-0.79
West-Central Section 24				
24009	230	**		
24053	479	**		
East-Southeast Section 24				
24064	848	**		
24065	989	**		
24088	1193	**		
24094	1207	**		
24095	1386	**		
24097	1065	**		
24098	969	**		
24100	757	**		
24105	1336	**		
Barrier Wells-Upgradient				
23120	321	5141.25(b)	5142.05	-0.08
23121	332	5141.5(b)	5141.59	-0.09
23123	444	5141.5(b)	5141.95	-0.45
23150	189	**		
24023	562	**		
24056	544	5141.25(b)	5141.58	-0.33
24058	537	5141.9(b)	5142.25	-0.35
24180	971	**		
Barrier Wells-Downgradient				
23043	740	**		
23196	373	**		
24006	799	5136.8(b)	5135.91	0.89
24163	1028	**		
Off Post-North				
37308	786	**		
37309	837	**		
37313	667	**		

(1). Node Number - Pilot System Mesh
(a). From: "Basin "F" to North Boundary. RMA, Denver, Colorado". Zebel 1979
(b). From: "Evaluation North Boundary Pilot Containment System", Battelle Columbus, 1979.
** No observed data reported for this period.

Table 3.5 Observed vs. Model Calculated Heads August 1981

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	5143.97	5143.94	0.03
23006	25	5143.93	5143.82	0.11
23007	48	5143.66	5143.62	0.04
23049	17	**		
23096	102	5143.17	5143.03	0.14
Central Section 23				
23004	223	5142.56	5142.43	0.13
23013	98	5143.35	5143.19	0.16
23014	67	5143.63	5143.48	0.15
23050	120	5143.33	5143.19	0.14
23051	82	5143.04	5143.03	0.01
West-Central Section 24				
24009	230	5142.38	5142.7	-0.32
24053	479	**		
East-Southeast Section 24				
24064	848	**		
24065	989	5152.85	5153.82	-0.97
24088	1193	5159.61	5162.12	-2.51
24094	1207	5154.98	5156.25	-1.27
24095	1386	5155.01	5157.84	-2.83
24097	1065	5148.50	5149.65	-1.15
24098	969	5147.03	5147.43	-0.04
24100	757	5143.55	5144.61	-1.06
24105	1336	**		
Barrier Wells-Upgradient				
23120	321	5140.12	5139.15	0.97
23121	332	**		
23123	444	5140.85	5139.70	1.15
23150	189	5142.32	5142.01	0.31
24023	562	5141.61	5141.04	0.57
24056	544	5138.59	5139.40	-0.81
24058	537	**		
24180	971	**		
Barrier Wells-Downgradient				
23043	740	**		
23196	373	**		
24006	799	5131.71	5134.41	-2.70
24163	1028	**		
Off Post-North				
37308	786	5124.42	5122.60	1.82
37309	837	**		
37313	667	5102.86	5102.33	0.53

(1). Node Number - Pilot System Mesh
(2). From: RMA Water Level Statistics and Plots. D.P. Assoc., Inc. Aug 1985.
** No observed data reported for this period.

Table 3.6 Observed/Interpolated vs Model Calculated Heads, November 1981

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	**		
23006	25	±5143.75	5143.80	-0.05
23007	48	5143.50	5143.65	-0.15
23049	17	±5144.	5143.86	0.14
23096	102	**		
Central Section 23				
23004	223	5142.74	5142.35	0.39
23013	98	±5143.25	5143.11	0.14
23014	67	5143.53	5143.42	0.11
23050	120	±5143.25	5143.11	0.14
23051	82	±5143.	5142.96	0.04
West-Central Section 24				
24009	230	5142.78	5142.61	0.17
24053	479	**		
East-Southeast Section 24				
24064	848	**		
24065	989	**		
24088	1193	±5159.5	5162.12	-2.62
24094	1207	±5155.	5156.26	-1.26
24095	1386	±5155.	5157.85	-2.85
24097	1065	±5149.	5149.66	-0.66
24098	969	±5147.5	5147.43	0.07
24100	757	5144.52	5144.58	-0.06
24105	1336	**		
Barrier Wells-Upgradient				
23120	321	**		
23121	332	**		
23123	444	±5141.25	5139.54	1.71
23150	189	±5142.25	5141.95	0.30
24023	562	**		
24056	544	**		
24058	537	**		
24180	971	**		
Barrier Wells-Downgradient				
23043	740	**		
23196	373	**		
24006	799	**		
24163	1028	**		
Off Post-North				
37308	786	±5122.5	5122.50	<0.01
37309	837	**		
37313	667	**		

(1). Node Number - Pilot System Mesh
(2). From: RMA Water Level Statistics and Plots. D.P. Assoc., Inc. Aug 1985.
** No observed data reported for this period.
(±) Signifies observed value interpolated from adjoining data and rounded to nearest quarter foot. To provide a measure of calibration at full barrier startup.

The agreement between model calculated and observed heads for East-Southeast Section 24 reflects the uncertainty in modeling transient interaction between First Creek and the aquifer. Wells 24088 and 24095 are within a short distance (<350 ft.) of a drainage ditch. The interaction between this ditch and the aquifer is undocumented. This ditch could serve as both local recharge and discharge sources depending upon the relative position of water flowing in the ditch and the aquifer. The water levels in these two wells is observed to rise and fall in relative synchrony throughout the recorded history. This may indicate seasonal variations in water levels within the ditch.

Transient Calibration: November 1981

Table 3.6 indicates the same general degree of comparison for November as was indicated for August 1981. Wells within a short distance of the pilot barrier show excellent agreement with the exception of well 23123. Wells 24088 and 24095 show the greatest difference between model calculated and observed heads, presumably due to the reasons postulated for August 81. Overall, the model is indicated to be accurate within ±3ft. and on the average, within ±0.65 ft. Generally this level of calibration is exceptional considering the complexity and dynamism of the alluvial aquifer system.

Over the period, Feb-March 1978 to November 1981, a regional decline in potentiometric levels on the order of 1-2ft. reflected a reduction in inflow to the model area and north boundary from approximately 255gpm under the pre-barrier calibration conditions to approximately 210gpm in November of 1981 when the full barrier system was put in operation. It is presumed that this decline may be due to a cessation of disposal of liquids in lined and

unlined basins to the south (up-gradient) of the model area in the late 1970's.

Full Barrier System: (November 1981 to Present Day)

Estimation of Individual Well Pumping and Recharge Rates:

As previously mentioned in section 3.3.a of this report, approximately 32 months of full barrier system operation data was recorded for the five years, and seven months of operation of the full barrier from November of 1981 to the present day. Originally only nine months of data was available that presented individual pumping and recharge rates for the 35 discharge , and 38 recharge wells screened in alluvium. The quality of this data is very poor due to mechanical and electrical failures of the flow metering. Assuming that the adsorber flow-through data was most representative of the operation of the barrier system, information concerning the distribution of the total manifold flows to the pumping and recharge wells was needed to model the aquifer response to barrier operation. Attempts to use individual well pumping information to allocate the manifold flows did not yield results that were at all acceptable.

Allocation of total manifold flows according to the model computed steady-state flux to the barrier resulted in an improved match for monitoring wells some distance away from the barrier system (south and central sections 23 and 24). Under this scheme, the recorded manifold flow rates were allocated to the individual wells based upon the ratio of the natural equilibrium flux to each well and the total model computed equilibrium flux to the manifold. Distribution to each recharge well was performed in a similar fashion for the sum of manifold pumping. This

methodology was utilized under the assumption that individual well pumping rates would be somewhat controlled by the natural flux to that region.

As previously stated, this method resulted in improvements in model computed heads vs observed heads for wells greater than 1000 ft. from the barrier system, and only minor improvement in the solution adjacent to the barrier. This gave good indication that the recorded manifold flux rates were representative of barrier operations, but the distribution of the flux rates across each manifold had not been accurately estimated.

Careful review of potentiometric mapping provided in "North Boundary Containment/Treatment System" report (RMA,1985) indicated that in the vicinity of the barrier system, similar drawdown patterns were indicated for maps prepared based upon February, May, and August of 1984. Similar patterns of relative potentiometric position were observed despite large fluctuations in manifold flow rates. This implied that the distribution of extraction and recharge remained relatively constant over this period. An equilibrium simulation was performed by setting the pumping and recharge wells to constant heads interpolated from the August 1984 potentiometric surface map (RMA,1985). In this way, the flow to each well that produced the August 1984 pattern, was computed. The net barrier extraction determined via this simulation was approximately 230gpm. This computed extraction rate compares well with the average total manifold flow rate of 250-260 gpm recorded for late August 1984. Computed net recharge agreed to within 20gpm of the total extraction rate. The individual contribution of each well was computed as a proportion of the total manifold flows.

The proportion factors were input into the model and for each reported change in manifold pumping, manifold rates were read in and allocated to each pumping and recharge well based upon the August 1984 model computed

proportion factors (see Tables 3.7 and 3.8). Where manifold flow data was unavailable, the average manifold rates recorded for the record to date were utilized.

A total of six comparisons of observed vs. model computed potentiometric data for full-barrier operation are included in this report (Tables 3.9 through 3.14). Comparisons for February, May, and August 1984 as well as April of 1986, and May of 1978, correspond to periods of time where manifold flow data was available. The data for July of 1985 represents the last period when comprehensive quarterly water level monitoring was performed. No system operation records were available for the ten months prior to July 1985. The May 1987 date corresponds to potentiometric level data collected in a cooperative effort between Colorado State University and the Rocky Mountain Arsenal on 13 May 1987.

On the average, the results of transient calibration and verification for the full barrier indicate equal or greater agreement with monitoring data than that for the pilot barrier system. For the five and one-half years of full barrier operation, more than one-hundred and fourty-eight individual observed vs model calculated comparisons are provided. On the average, the model computed heads are within $\pm$0.73ft. of that observed. The average error for thirty-eight points of recorded data for the twelve monitoring wells immediately adjacent to the barrier is $\pm$0.72feet. This level of performance is considered exceptional given the sensitivity of these wells to barrier operations and the degree of uncertainty associated with the barrier operation data.

Table 3.7 Model Computed Flow Proportioning and Average Rates Discharge Wells

1 Well Number	2 Proportion	3 Pumping @ Average Manifold Flow Rate
Manifold A, 21 Month Average Flow Rate = 63gpm (4)		
330	0.0335	2.11
331	0.0169	1.06
332	0.0367	2.31
333	0.0093	0.59
334	0.0055	0.35
335	0.0037	0.23
301	0.0303	1.91
302	0.0426	2.68
303	0.0	0.0
304	0.2999	18.89
305	0.2001	12.61
306	0.3216	20.26
307	0.0	0.0
Manifold B, 21 Month Average Flow Rate = 75gpm (4)		
308	0.0	0.0
309	0.0	0.0
310	0.0	0.0
311	0.0	0.0
312	0.0	0.0
313	0.2227	16.7
314	0.2433	18.25
315	0.098	7.35
316	0.1455	10.91
317	0.1006	7.55
318	0.1899	14.24
Manifold C, 21 Month Average Flow Rate = 73gpm (4)		
319	0.0797	5.82
320	0.0	0.0
321	0.1863	13.60
322	0.1934	14.12
323	0.2385	17.41
324	0.0520	3.8
325	0.1128	8.23
326	0.1046	7.64
327	0.0277	2.02
328	0.0	0.0
329	0.0050	0.37

Notes:
1. RMA Well Number
2. Based upon August 1984 calibration run
3. Average Well Discharge - based upon proportion times average manifold flow rate.
4. Average manifold flow rate computed based upon the 21 months of record available to this date.

Table 3.8 Model Computed Flow Proportioning and Average Rates
Recharge Wells

1 Well Number	2 Proportion	3 Recharge @ Average Manifold Flow Rate
21 Month Average Recharge Rate(sum of manifold rates)= 211gpm		
401	0.002	-0.42
402	0.0032	-0.68
403	0.0091	-1.92
404	0.0081	-1.71
405	0.0	0.0
406	0.0245	-5.17
407	0.1035	-21.84
408	0.0283	-5.97
409	0.0341	-7.20
410	0.0277	-5.84
411	0.0031	-0.65
412	0.0001	-0.02
413	0.0	0.0
414	0.0	0.0
415	0.0277	-5.84
416/bog	0.0528	-11.14
417/bog	0.0401	-8.46
418/bog	0.0777	-16.39
419/bog	0.0525	-11.08
420/bog	0.0331	-6.98
421	0.0126	-2.66
422	0.1791	-37.79
423	0.0824	-17.39
424	0.0628	-13.25
425	0.0283	-5.97
426	0.0644	-13.59
427	0.0142	-3.00
428	0.0026	-0.55
429	0.0029	-0.61
430	0.0	0.0
431	0.0	0.0
432	0.0106	-2.24
433	0.0043	-0.91
434	0.0	0.0
435	0.0	0.0
436	0.0026	-0.55
437	0.004	-0.84
438	0.0085	-1.79

Notes: 1. RMA designated well number.
2. Proportion based upon August 1984 calibration run.
3. Average Well Recharge Rate= sum of manifold rates times proportion factor column 2.

Table 3.9 Observed vs. Model Calculated Heads, February 1984

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	5143.92	5144.10	-0.18
23006	25	5143.90	5143.96	-0.06
23007	48	5143.67	5143.71	-0.04
23049	17	**		
23096	102	**		
Central Section 23				
23004	224	5142.94	5142.69	0.25
23013	98	**		
23014	67	5143.25	5143.51	-0.26
23050	120	5142.56	5143.11	-0.55
23051	82	5142.92	5143.10	-0.18
West-Central Section 24				
24009	231	5142.53	5142.91	0.38
24053	364	**		
East-Southeast Section 24				
24064	504	5152.74	5151.36	1.38
24065	581	5154.83	5154.61	0.22
24088	695	5163.24	5163.06	0.18
24094	700	5158.09	5157.02	1.07
24095	787	5159.53	5158.53	1.00
24097	626	5150.71	5150.48	0.23
24098	573	5149.38	5148.31	1.07
24100	461	5145.32	5145.46	-0.14
24105	734	**		
Barrier Wells-Upgradient				
23120	291	5140.29	5139.96	0.33
23121	287	**		
23123	444	5140.76	5140.43	0.33
23150	189	5141.58	5141.3	0.28
24023	398	5142.32	5141.78	0.54
24056	391	**		
24058	387	**		
24180	561	**		
Barrier Wells-Downgradient				
23043	1259	**		
23196	980	**		
24006	1200	Dry, <5129.43	5131.02	NA
24163	1404	**		
Off Post-North				
37308	1272	**		
37309	1274	**		
37313	1168	**		

(1). Node Number - Full Barrier Mesh
(2). From: RMA Water Level Statistics and Plots, August 1985. Volumes I and II, D.P. Associates, Inc., August 1985.
** No observed data reported for this period.

Table 3.10 Comparison May 1984

Table 3.10 Observed vs. Model Calculated Heads, May 1984

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	5143.89	5144.68	-0.79
23006	25	5143.97	5144.59	-0.62
23007	48	5143.57	5144.38	-0.81
23008	101	5144.08	5145.25	-1.17
23049	17	**		
23096	102	5143.51	5143.96	-0.45
Central Section 23				
23004	224	5143.24	5143.60	-0.36
23013	98	5144.07	5143.88	0.19
23014	67	5143.15	5144.18	-1.03
23050	120	5143.26	5143.81	-0.55
23051	82	5143.02	5143.89	-0.87
West-Central Section 24				
24009	231	5143.53	5143.81	-0.28
24053	364	**		
East-Southeast Section 24				
24064	504	5153.54	5151.95	1.59
24065	581	5155.83	5155.01	0.82
24088	695	5163.44	5163.27	0.17
24094	700	5158.79	5157.31	1.48
24095	787	5159.53	5158.73	0.80
24097	626	5151.81	5151.03	0.78
24098	573	5150.08	5148.97	1.11
24100	461	5145.82	5146.24	-0.42
24105	734	5142.98	5147.17	-4.19
Barrier Wells-Upgradient				
23120	291	5140.99	5140.51	0.48
23121	287	**		
23123	444	5141.56	5141.01	0.55
23150	189	5141.88	5142.03	-0.15
24023	398	5142.72	5142.58	0.14
24056	391	**		
24058	387	**		
24180	561	±5140.5*	5141.64	NA
Barrier Wells-Downgradient				
23043	1259	5131.54-5131.74*	5129.66	NA
23196	980	**		
24006	1200	5133.25-<5129.43*	5129.82	NA
24163	1404	5131.69- 5135.84*	5133.58	NA
Off Post-North				
37308	1272	5124.89- 5123.89*	5121.07	NA
37309	1274	**		
37313	1168	5107.76- 5107.06*	5106.36	NA

(1). Node Number - Full Barrier Mesh

(2). From: RMA Water Level Statistics and Plots, August 1985. Volumes I and II, D.P. Associates, Inc., August 1985.

* Average or range of values presented for comparison purposes.

** No observed data reported for this period.

NA Head difference not applicable, presented for comparison only.

Table 3.11 Comparison August 1984

Table 3.11 Observed vs. Model Calculated Heads, August 84

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	5144.29	5144.52	-0.23
23006	25	5143.77	5144.37	-0.60
23007	48	5143.37	5144.12	-0.75
23008	101	**		
23049	17	5145.04	5144.45	0.59
23096	102	**		
Central Section 23				
23004	224	5142.74	5142.28	0.48
23013	98	5144.07	5143.56	0.51
23014	67	5143.15	5143.93	-0.78
23050	120	5143.26	5143.55	-0.29
23051	82	5143.02	5143.63	-0.61
West-Central Section 24				
24009	231	5143.53	5142.73	0.80
24053	364	**		
East-Southeast Section 24				
24064	504	5152.94	5151.12	1.82
24065	581	5155.43	5154.47	0.96
24088	695	5162.94	5163.15	-0.21
24094	700	5158.59	5156.97	1.62
24095	787	5158.43	5158.56	-0.13
24097	626	5151.51	5149.97	1.54
24098	573	5149.78	5147.61	2.17
24100	461	5145.32	5144.6	0.72
24105	734	**		
Barrier Wells-Upgradient				
23120	291	5139.99	5139.42	0.57
23121	287	**		
23123	444	5140.56	5139.84	0.72
23150	189	5141.88	5142.0	-0.12
24023	398	5142.12	5141.22	0.90
24056	391	**		
24058	387	**		
24180	561	±5138.5*	5139.33	NA
Barrier Wells-Downgradient				
23043	1259	**		
23196	980	**		
24006	1200	5134.15	5132.32	1.83
24163	1404	5135-5134*	5134.28	NA
Off Post-North				
37308	1272	**		
37309	1274	**		
37313	1168	**		

(1). Node Number - Full Barrier Mesh

(2). From: RMA Water Level Statistics and Plots, August 1985. Volumes I and II, D.P. Associates, Inc., August 1985.

* Average or range of values presented for comparison purposes.

** No observed data reported for this period.

NA Head difference not applicable, presented for comparison only.

Table 3.12 Observed vs. Model Calculated Heads, July 1985

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	5143.89	5144.38	-0.49
23006	25	5143.77	5144.22	-0.45
23007	48	5143.67	5143.97	-0.30
23008	101	5143.98	5145.21	-1.23
23049	17	**		
23096	102	5143.01	5143.21	-0.20
Central Section 23				
23004	224	5142.34	5142.49	-0.15
23013	98	5144.07	5143.37	0.70
23014	67	5143.45	5143.76	-0.31
23050	120	5142.86	5143.35	-0.49
23051	82	5142.82	5143.21	-0.39
West-Central Section 24				
24009	231	5142.43	5142.81	-0.38
24053	364	5143.29	5143.1	0.19
East-Southeast Section 24				
24064	504	5152.04	5151.08	0.96
24065	581	5153.83	5154.46	-0.63
24088	695	5161.04	5163.10	-2.06
24094	700	5156.09	5156.95	-0.86
24095	787	5156.33	5158.53	-2.20
24097	626	5149.21	5150.07	-0.86
24098	573	5147.48	5147.74	-0.26
24100	461	5143.52	5144.71	-1.19
24105	734	**		
Barrier Wells-Upgradient				
23120	291	5138.19	5138.69	-0.50
23121	287	5138.4	5139.29	-0.89
23123	444	5139.26	5139.26	<0.01
23150	189	5141.68	5141.4	0.28
24023	398	5140.82	5140.94	-0.12
24056	391	5139.09	5139.19	-0.10
24058	387	5139.86	5140.07	-0.21
24180	561	5138.32	5139.85	-1.53
Barrier Wells-Downgradient				
23043	1259	5131.84	5132.54	-0.70
23196	980	5123.83	5123.49	0.34
24006	1200	DRY,<5129.43*	5132.77	NA
24163	1404	5134.69	5135.43	-0.74
Off Post-North				
37308	1272	5123.37	5122.90	0.47
37309	1274	5119.73	5117.25	2.48
37313	1168	5106.66	5106.38	0.28

(1). Node Number - Full Barrier Mesh
(2). From: RMA Water Level Statistics and Plots, August 1985. Volumes I and II, D.P. Associates, Inc., August 1985.
* Average or range of values presented for comparison purposes.
** No observed data reported for this period.
NA Head difference not applicable, presented for comparison only.

Table 3.13 Observed vs. Model Calculated Heads, April 1986

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	**		
23006	25	**		
23007	48	**		
23008	101	**		
23049	17	**		
23096	102	**		
Central Section 23				
23004	224	**		
23013	98	**		
23014	67	**		
23050	120	**		
23051	82	**		
West-Central Section 24				
24009	231	**		
24053	364	**		
East-Southeast Section 24				
24064	504	**		
24065	581	**		
24088	695	**		
24094	700	**		
24095	787	**		
24097	626	**		
24098	573	**		
24100	461	**		
24105	734	**		
Barrier Wells-Upgradient				
23120	291	5139.07	5138.25	0.82
23121	287	**		
23123	444	**		
23150	189	5141.39	5141.09	0.30
24023	398	**		
24056	391	**		
24058	387	**		
24180	561	**		
Barrier Wells-Downgradient				
23043	1259	5132.34	5132.58	-0.24
23196	980	5123.04	5123.74	-0.70
24006	1200	**		
24163	1404	5133.51	5134.54	-1.03
Off Post-North				
37308	1272	**		
37309	1274	**		
37313	1168	**		

(1). Node Number - Full Barrier Mesh
(2). From: RMA Water Elevation Report For the North Boundary System: April 1985 Through May 1986, D.P.Associates, Inc. 1986.
* Average or range of values presented for comparison purposes.
** No observed data reported for this period.
NA Head difference not applicable, presented for comparison only.

Table 3.14 Observed vs. Model Calculated Heads, May 13, 1987

Well No.	(1) Node No.	(2) Observed Head (ft. AMSL)	(3) Calculated Head (ft. AMSL)	(4)= (2) - (3) Head Difference (ft.)
South Section 23				
23003	16	5143.0	5143.25	-0.25
23006	25	5143.03	5143.01	-0.02
23007	48	5142.69	5142.72	-0.03
23008	101	5143.76	5144.85	-1.09
23049	17	5143.46	5143.11	0.35
23096	102	5142.24	5141.71	0.53
Central Section 23				
23004	224	5141.62	5140.92	0.70
23013	98	5143.23	5142.07	1.16
23014	67	5142.8	5142.48	0.32
23050	120	5142.18	5142.08	0.10
23051	82	5141.97	5141.78	0.19
West-Central Section 24				
24009	231	5142.89	5141.24	1.65
24053	364	5141.79	5141.28	0.51
East-Southeast Section 24				
24064	504	5151.83	5150.	1.83
24065	581	5153.94	5153.65	0.28
24088	695	5162.1	5162.72	-0.62
24094	700	5157.09	5156.38	0.71
24095	787	5158.64	5158.15	0.49
24097	626	5149.82	5148.83	0.99
24098	573	5147.91	5146.16	1.75
24100	461	5143.39	5142.77	0.62
24105	734	5142.68	5143.65	-0.97
Barrier Wells-Upgradient				
23120	291	5138.84	5137.94	0.90
23121	287	5139.07	5138.36	0.71
23123	444	5139.96	5138.28	1.68
23150	189	**		
24023	398	5141.03	5139.29	1.74
24056	391	5139.63	5138.17	1.46
24058	387	5140.12	5138.75	1.37
24180	561	**		
Barrier Wells-Downgradient				
23043	1259	5131.45	5133.55	-2.10
23196	980	5122.55	5124.26	-1.71
24006	1200	DRY, 5129.43	5133.77	NA
24163	1404	5135.69	5135.63	0.06
Off Post-North				
37308	1272	**		
37309	1274	**		
37313	1168	**		

(1). Node Number - Full Barrier Mesh
(2). Observed potentiometric levels from depth to groundwater level measurements performed 13, May 1987 in a cooperative effort between CSU and the RMA.
** No observed data reported for this period.
NA Head difference not applicable, presented for comparison only.

For each of the six sets of comparison data, the greatest errors are recorded for monitoring wells within East-SouthEast Section 24, adjacent to First Creek or drainage features of First Creek.
Wells 24105, 24098, 24095 and 24088 are all less than a few hundred feet from branches of First Creek, or the RMA sanitary sewer outfall. Potentiometric levels for these wells may be influenced by local recharge/discharge relations of the surface features.

As with the pilot barrier system, monitoring wells immediately adjacent to the barrier system are quite sensitive to small variations in operation of the barrier. This is particularly true for down-gradient wells 23043, 24006, 24163, and 23196. Except for one observation for well 23196, the error is generally less than ± 1.5ft.

Comparison of observed vs model calculated heads for May 13, 1987 is provided in Table 3.14. At this point in the calibration the entire model was accurate to within ± 2 feet., the exception being well 23043. On the average for this date, the model is within ± 0.86 feet (≈ 10 inches) at the thirty-one monitoring points.

Recalibration:

Most of the water level data collected up to this point in the calibration process reflected head conditions upgradient of the barrier. However, in 1987 additional monitoring wells were installed upgradient and downgradient of the barrier by Environmental Science and Engineering (1987) providing useful data for refinement of the model. It was decided that the focus of additional calibration efforts would be on the downgradient side of the barrier for the following reasons:

1) The degree of calibration in this areas was not well defined due to lack of observed data, and;

2) Most of the model simulations would involve reconfiguration or augmentation schemes downgradient of the barrier (e.g. trenches, additional wells) and thus mesh refinement in this area was needed.

In order to refine model results and to be capable of representing recharge system reconfiguration or augmentation schemes, another two line of nodes between the recharge wells and the barrier were added to the mesh. The original mesh for the full-barrier system consisted of 1632 nodes and 2980 elements. The present full-barrier mesh consists of 1720 nodes and 3152 elements. The addition of these new nodes and elements allows for more model refinement in a sensitive area immediately downgradient of the slurry wall (within 500 feet) and thus allowing for better calibration downgradient and increased capability of modeling the system operation.

Table 3.15 summarizes the recalibration effort results. It lists the differences between observed and model calculated heads for the new data obtained from ESE (downgradient monitoring wells). The table indicates that the worst match is near the west end where the water table is very steep. The average difference here is approximately two feet. However, it is estimated that this area accounts for less than three percent of total barrier system flow. The average difference for all other downgradient monitoring wells used in calibration is 1.07 feet. This is exceptional considering the fact that most of the wells are located in a very sensitive area close to the barrier.

Table 3.15 Observed vs. Model Calculated Heads, May 13, 1987
(New Data)

Well No.	(1) Node No.	(2) Observed Head	(3) Calculated Head	(4)= (3) - (2) Head Difference
West End				
23039	1001	5118.90	5118.20	0.70
23040	973	5130.20	5127.30	2.86
23196	993	5122.60	5123.99	-1.39
23197	998	5125.40	5128.88	-3.48
23148	949	5141.10	5138.45	2.65
23205	959	5139.10	5136.89	2.21
23197	998	5134.40	5128.88	-3.48
23198	1009	5127.50	5129.08	-1.58
23124	988	5134.10	5132.70	1.41
23166	984	5134.00	5133.94	0.06
23215	979	5131.40	5132.57	-1.17
37339	999	5121.30	5124.20	-2.90
37306	1016	5127.00	5123.69	3.31
Near Pilot Sys.				
23048	1034	5127.20	5127.72	-0.52
23047	1074	5126.80	5127.47	-0.67
23110	1100	5127.90	5127.78	0.12
23046	1128	5126.70	5128.20	-0.22
23216	1092	5128.40	5129.41	-1.01
37307	1152	5127.40	5126.18	1.22
23045	1205	5128.50	5128.72	-0.22
23044	1250	5131.00	5128.42	2.58
23217	1257	5132.60	5132.00	0.60
23043	1296	5131.00	5129.93	1.07
37311	1318	5130.60	5129.52	1.08
24192	1336	5132.10	5133.22	-1.12
24169	1352	5133.10	5133.75	-0.65
23111	1236	5130.50	5130.79	-0.29
Near the Bog				
24161	1350	5132.00	5133.54	-1.54
37312	1361	5133.20	5132.50	0.70
24162	1375	5133.40	5132.61	0.79
24193	1400	5134.70	5135.24	-0.54
24163	1460	5134.80	5134.93	-0.13
24194	1453	5136.60	5135.96	0.64
24170	1471	5138.30	5135.98	2.32
24164	1511	5135.60	5133.95	1.65
24195	1525	5136.70	5135.95	0.75
Near First Creek				
37338	1567	5129.70	5129.28	0.42
24166	1629	5131.80	5131.10	0.70
24176	1684	5135.30	5136.67	-1.37
37362	1692	5129.70	5130.42	-0.73
37389	1404	5123.80	5125.75	-1.95

Table 3.15 Observed vs. Calculated Head, May 13, 1987
(Continued)

Well No.	(1) Node No.	(2) Observed Head	(3) Calculated Head	(4)= (3) - (2) Head Difference
Off-Post Northeast				
37377		5112.40	5114.93	-2.532
Near First Creek Imp.				
37313	1198	5106.20	5106.26	-0.61
37381	1139	5105.40	5108.44	-3.04
37370	1116	5110.40	5111.96	-1.56
37374	1097	5108.90	5107.58	1.32
37373	1194	5109.20	5110.69	-1.49
37343	1230	5106.30	5106.96	-0.66
37308	1312	5123.10	5121.43	1.66
37309	1303	5119.50	5118.86	0.64
37369	1255	5120.10	5118.63	1.47

It should be noted that the best match may be found near the location of the original pilot scale system with the exception of well 23044. Leakage from the effluent pipe used to convey water from the treatment plant to the recharge wells has been observed by operations personnel in the area near this well thus a groundwater mound is suspect in this area. The results of calibration near the pilot scale system are particularly enlightening considering the fact that this part of the barrier intercepts the highest concentration of contaminated water. Model accuracy is of utmost importance in this area.

The computed heads for the area near the bog (used for groundwater recharge) also have a relatively good agreement with observed data. The bog was originally modeled as point recharge through four recharge wells located at nodal points in or near the bog. However, better agreement was attained when the bog was modeled as a constant head area. Using the model a constant head of 136 feet was found to render the best results, which agrees with the surveyed head of 136 feet taken in Spring 1987 (ESE).

The new observed water level data and bog recharge data obtained from RIC also made it apparent that the model transmisivities were too low in the area near the bog. The data indicated that approximately 55 percent of the total system recharge was through the bog. The original model estimation was approximately 30 percent. Better agreement was attained when model transmissivites were raised approximately 20 percent. The present model estimation for recharge through the bog is 49 percent of the total system recharge.

It should be noted that in a few cases, the exact location of the new monitoring wells could not be matched by a node. In these cases, the closest node to the well location was used to represent the well. During

calibration, a value for the exact well location was interpolated between surrounding nodes. In most cases, this resulted in an improved agreement between computed and observed values.

In addition to the additional monitoring well data collected, data on the operation of the barrier system had been tabulated by The Rocky Mountain Arsenal Information Center (RIC). This data included observed recharge and discharge well rates, manifold pump rates, and adsorber rates for the period of operation from July 1985 to the present day. Average values for the 21 months of data were utilized as a starting point in calibration for calibrating to May 1987 water level conditions. The calibration effort primarily involved adjusting these average recharge well rates to attain a suitable match between model computed water levels and observed water levels in May 1987.

Previously it was stated that the flow metering system for the recharge and discharge wells was not accurate enough to render useful data. This was especially true for the period of operation before 1985. However, recent management and tabulation of the barrier system operation data has strengthened the usefulness of the data. The well rates provided in the RIC data provide at least an indication of relative flow rates of the wells. Consequently, it was decided that the individual well rates could be used as a starting point for calibration and adjusted to obtain a suitable match between model calculated and observed water table heads downgradient. After a suitable match was obtained, the adjusted well rates would be viewed as the historical average recharge rates for each well. Table 3.16 lists the average recharge rates and proportion factors for each well obtained from RIC data and rates and proportion factors calculated through calibration of

Table 3.16 Recharge Well Rates

Well No.	Average Prop Factor	Rate (μ)	Calibrated Prop Factor	Rate (GPM)	Diff (μ)
401	0.0022	0.54	0.0009	0.22	0.32
402	0.0015	0.37	0.0015	0.37	0.00
403	0.0052	0.27	0.0052	1.27	0.00
404	0.0067	1.64	0.0067	1.64	0.00
405	0.0064	1.57	0.0040	0.98	0.59
406	0.0206	5.05	0.0020	0.49	4.56
407	0.0038	0.93	0.0020	0.49	0.44
408	0.0124	3.04	0.0120	2.94	0.10
409	0.0106	2.60	0.0110	2.70	-0.10
410	0.0035	0.86	0.0205	5.02	-4.16
411	0.0035	0.86	0.0140	3.43	-2.57
412	0.0030	0.74	0.0050	1.22	-0.48
413	0.0117	2 87	0.0121	2.96	-0.09
414	0.0111	2.72	0.0080	1.96	0.76
415	0.0068	1.67	0.0070	1.72	-0.05
416	0.0123	3.01	0.0300	7.35	-3.77
417	0.0248	6.08	*	*	-----
418	0.1029	25.21	*	*	-----
419	0.1362	33.37	*	*	-----
420	0.1735	42.51	*	*	-----
421	0.1735	42.51	*	*	-----
422	0.0222	5.44	0.0500	12.25	-6.81
423	0.0158	3.87	0.4520	11.08	-7.21
424	0.0180	4.41	0.0300	7.35	-2.94
425	0.0521	12.76	0.0800	19.60	-6.84
426	0.0213	5.22	0.0450	11.02	-5.80
427	0.0370	9.06	0.0700	17.15	-8.09
428	0.0089	2.18	0.0151	3.70	-1.52
429	0.0098	2.40	0.0120	2.94	-0.54
430	0.0289	7.08	0.0300	7.35	-0.27
431	0.0304	7.45	0.0304	7.45	-0.00
432	0.0167	4.09	0.0060	1.47	2.62
433	0.0011	0.27	0.0047	1.15	-0.88
434	0.0160	3.92	0.0001	0.02	3.90
435	0.0049	1.20	0.0002	0.05	1.15
436	0.0010	0.24	0.0039	0.96	-0.72
437	0.0032	0.78	0.0020	0.49	0.29
438	0.0095	2.33	0.0018	0.44	1.89

RATES BASED ON A 21 MONTH AVERAGE RECHARGE RATE OF 245 GPM

*Note: The bog was modeled as a constant head in this area.
Average recharge from the bog over 21 months = 143 GPM
Model computed recharge from the bog = 120 GPM

the model. Many of the well rates are less than 1 gpm away from the RIC averages. However a few of the calibrated rates are as far as 6 and 7 gpm away from the RIC Data. This discrepancy is attributed to the inability to accurately meter the flow rates of the recharge wells.

As stated before, calibration is particularly difficult in the area immediately surrounding the barrier due to the natural steep gradient and sensitivity of monitoring wells to recharge. Observed data provided by ESE for 49 additional monitoring wells downgradient of the site allowed for a clearer definition of the water table.

Table 3.17 presents the model computed flux rate to the north boundary for 13 May 1987. On this date, it is estimated that a total of 220gpm is flowing to the north boundary barrier system. For comparison, Table 3.17 presents the model computed flux rates to the north boundary at select times in the history of barrier operation. It should be noted as a result of review of Table 3.17, that the concept of a unique "steady" equilibrium flow rate to the north boundary is a fallacy and that the system should be viewed as dynamic, responding to variations in recharge or other phenomenon up-gradient of the model area.

It is interesting to compare the model computed natural interception manifold rates with the range and average of twenty months of full barrier operation data (Table 3.18). Despite a wide range of operation rates for each manifold, the average operation rates compare very closely with the interception rate computed for the present day. It is intuitive that the system would have to have been operated at the natural interception rate on the average for the period of record. Long-term operation at rates greater, or less than natural interception would result in local aquifer desaturation or flooding respectively.

Table 3.17 Comparison of Model Computed Groundwater Flow to the North Boundary at Select Dates.

Date	Computed Flow To North Boundary
Feb-March 1978	255gpm
Nov 1981	210gpm
May 13, 1987	220gpm

Table 3.18
Comparison of Recorded Manifold Operation Rates with Model Predicted Natural Interception, May 1987

	Observed		Model Computed
Manifold	Range (gpm)	Average	Natural
A	31 - 101	59gpm	55gpm
B	41 - 118	83gpm	70gpm
C	39 - 145	103gpm	95gpm
Total	136 - 314	245gpm	220gpm

Table 3.19 presents a complete summary of all comparisons of model computed versus observed potentiometric levels for transient calibration and verification. The number of comparisons, maximum difference, and average difference between model computed results and observed data for each of the total ten separate comparison periods are provided.

In summary, the results of transient calibration and verification are exceptional given the dynamism and complexity of the alluvial aquifer and groundwater interception system at the north boundary. The model was calibrated to February-March 1978 potentiometric information. The model was verified by running nine and one quarter years through installation and operation of the pilot and full barrier systems to the present day. For this 9.25 year period, ten separate comparisons of model computed vs observed potentiometric levels were compiled. The comparison data included thirty-six monitoring wells located throughout the model area. The model was then recalibrated from 1985 to the present day with new data for 49 additional monitoring wells downgradient of the barrier. A total of two-hundred fifty nine separate comparisons of model computed and observed levels were made. The average error (difference between model computed and observed) for the 259 individual comparisons is $\pm$0.77ft. In the vicinity of the barrier system, the average error is indicated to be $\pm$0.81ft. It is thought that this level of calibration is considered excellent. The model is deemed suitable for simulation of the effects of barrier operation strategies and reconfiguration concepts.

Table 3.19

Summary Transient Calibration and and Verification
North Boundary Containment Treatment System Model

Period	No. of Comparisons	Monitoring Well Observed vs. Model Predicted	
		Maximum Difference	Average
Pre-Barrier	10	0.74ft	0.29ft
Pilot System (July 78-Nov 81)	52	2.85ft	0.65ft
March 1979	9	0.89ft	0.39ft
August 1981	25	2.83ft	0.80ft
November 81	18	2.85ft	0.60ft
Present System (Nov 81 - Pres)	148	4.19ft	0.73ft
February 84	21	1.38ft	0.43ft
May 1984	30	4.19ft	0.80ft
August 1984	25	2.17ft	0.82ft
July 1985	35	2.48ft	0.67ft
April 1986	5	1.03ft	0.62ft
May 13 198	32	2.10ft	0.87ft
May 13 1987 Recalibration	49 (additional)	3.48ft	1.15ft
Total Recorded History (February 1978 through May 1987) ≈ 9.25yrs	259	4.19ft	0.78ft

CHAPTER 4

MODEL SIMULATIONS

Utilizing the calibrated and verified model, various operational and barrier reconfiguration simulations were performed. The simulations were selected to aid in answering questions posed by the Arsenal personnel in numerous discussions. Model simulations were of three basic types; operational simulations involving the existing barrier configuration, operation simulations involving barrier reconfiguration schemes and simulations involving breakdown of barrier system. Barrier reconfiguration could include, addition of new pumping and recharge wells, abandonment of the existing wells and establishment of new well arrays closer to the slurry wall, addition of recharge trenches and numerous other schemes. Most of the simulations were performed to estimate the results of barrier operation on the hydraulic gradient across the bentonite slurry wall. The Arsenal is interested in means by which the hydraulic gradient across the barrier could be minimized, or reversed from the natural pre-barrier northward gradient. Other simulations were performed to investigate operational breakdown conditions. The breakdown simulations were concerned with determining how long it takes for the groundwater to overtop the barrier and where it will overtop when a series of discharge wells fail or the entire treatment plant fails. These breakdown simulations will be discussed at the end of this chapter.

As previously discussed, limitations to approaching or attaining a gradient reversal include: recharge and discharge well spacing with respect

to each other and the slurry wall, and placement of the system in the natural pre-barrier hydrologic setting.

For the northwest boundary system, a gradient reversal was possible for the hydrologic control section as a result of over-pumping the hydrologic control wells and inducing recirculation of water. Review of information compiled for the northwest boundary study indicates that a reverse gradient for the slurry wall section was not always attained in operation of that system. The north boundary system differs in that the slurry wall forms a complete cut-off of the alluvial aquifer. A reverse gradient due to recirculation of water from the recharge wells is not possible. Long-term pumping of the system is limited to the natural flow rate intercepted by the barrier.

Results of operational simulations indicate that the potentiometric head observed on the upgradient, and downgradient sides of the slurry wall is typically within a few tenths of a foot of the head computed for the node representing the adjacent pumping or recharge well. This model computed result is in general agreement with observations of actual barrier operation (RMA, 1985). Under the existing barrier system, the recharge wells are an average of 500 ft horizontally downgradient of the discharge wells. Under the natural (Feb-Mar 1978) pre-barrier conditions the potentiometric surface declined an average of nearly four feet over this distance. Assuming that the long term operational pumping rate of the barrier system is limited to the equilibrium flow rate intercepted by the barrier, an average hydraulic gradient across the barrier on the order of four feet should be expected based on the Feb-March pre-system potentiometric levels. Some differences in hydraulic gradient should be observed since under natural conditions the

flow in the aquifer is distributed whereas under barrier operation the flow is concentrated as a point sink or source at the wells.

Shown on Figure 4.1 is the anticipated distribution of gradient across the barrier under the pre-barrier conditions assuming that the pre-barrier (Feb-March 1978) heads at the well locations is the head resulting at the slurry wall.

The excellent results obtained in the calibration and verification phase of the project indicated that the model is ready for simulation of operational management and barrier reconfiguration schemes. Many simulations were coded and run successfully. It should be pointed out, that the digital model is a means of obtaining an approximate solution to a very complex natural system.

Some of the limitations of modeling any natural system include:

1. A natural system is dynamic. Short and long-term trends in climactic factors may effect the term of applicability of model results. Changes in inflow to the model area were observed over the period of transient calibration and verification. Both distribution and magnitude of inflow changed over the nine and one quarter years modeled. The observed changes in inflow may be due to long-term effects of cessation of basin disposal in the late 1970's, or other unpredictable, and unquantified, factors. A stabilization of potentiometric hydrographs for monitoring wells located at the south (inflow) limits of the model for the last five years has been observed. Model predicted flux to the north boundary has varied approximately ten gallons per minute over this period. The majority of simulations were

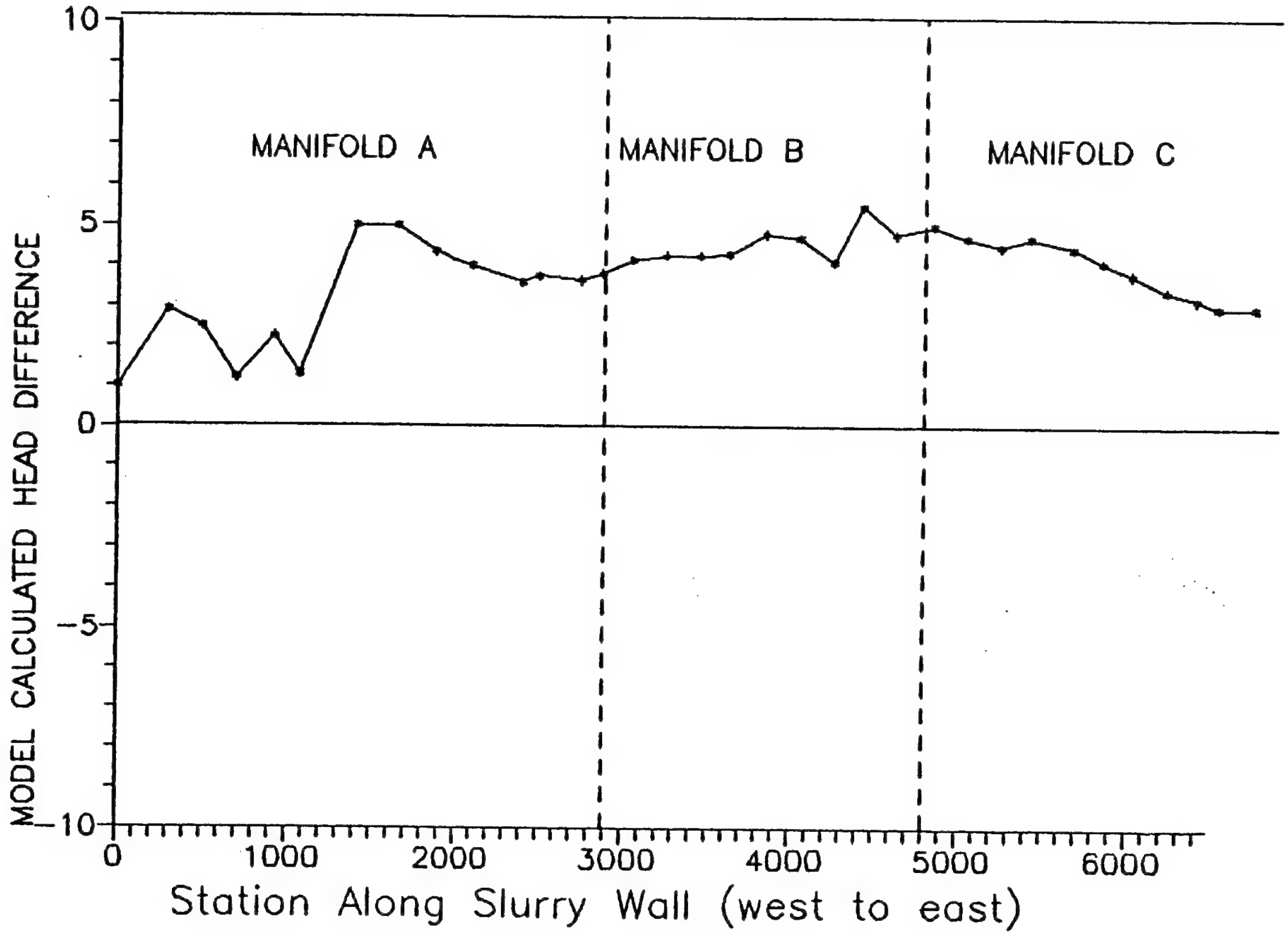


Figure 4.1 Natural Pre-barrier Gradient from Discharge to Recharge Well

performed assuming that the present day (May 1987) flux rates remain constant.

It should be recognized that present day remedial activities at the Arsenal could significantly change the direct applicability of these results. Activities which may have a bearing on groundwater flows to the north boundary include: containerization and abandonment of Basin "F" and installation of source, or near source interception systems such as is currently under design for the RMA immediately north of Basin "F".

2. The model was calibrated to pre-barrier groundwater conditions. Changes in the aquifer in the immediate vicinity of the slurry wall due to penetration of the bentonite slurry into the aquifer cannot be quantified. The exceptional results of transient calibration and verification for the vicinity of the barrier indicate that this is likely not a serious problem.

3. The model is meant to simulate the aquifer response to applied stresses such as barrier pumping and recharge. System constraints such as well losses, and clogging of well screens/gravel pack due carbon fines, cannot be explicitly modeled. The slurry wall barrier is modeled as an internal row of impermeable (no flow) boundary nodes.

4.1 <u>Operational Simulations</u>

These simulations determined the effect of current, historical, and hypothetical operational alternatives. They include pumping allocation based upon the historical manifold well proportions determined in the transient

calibration and verification phase, pumping allocation based upon natural equilibrium flux to the manifolds (May, 1987).

4.1.a Natural Interception, Natural Proportioning

This simulation consists of an evaluation of the present (May 1987) natural flow rate intercepted by the barrier manifolds. The individual well discharge and recharge rates listed in Table 4.1 were computed by the following scheme: The FEB-MAR pre-barrier model (natural conditions) was run at steady state using the boundary fluxes from the May 1987 data set. This in effect, produced the natural gradient that would be calculated in 1987, if the barrier did not exist. The gradient between the location of the rows of discharge and recharge wells resulting from this simulation was transposed onto the May 1987 model. Another steady state simulation was performed using this model and setting the transposed gradient to a constant value to give the effect of a natural gradient in 1987. The constant gradient was simulated by setting the pumping and recharge well nodes to constant values. By setting these nodes to constant values, it was possible to calculate the flow to these nodes that was needed to maintain this natural gradient. The resulting flow rates are the natural equilibrium flux rates to the respective wells for May 1987.

Under the existing (May 87) natural equilibrium pumping and recharge rates, the average head differentials across the slurry wall barrier are estimated to be 2.5, 4.0, and 4.0 feet for manifolds A, B, and C respectively. A positive gradient across the slurry wall by convention, would be one in keeping with the pre-barrier natural northward gradient at the barrier. The model computed distribution of the differential is shown on

Table 4.1 Natural Interception Pumping and Recharge Rates-Simulation 4.1.a

Discharge Wells		Recharge Wells	
Number	Rate (gpm)	Number	Rate (gpm)
Manifold A	55gpm total	Total Recharge	-220
330	2	432	-1
331	2	433	-1
332	1	434	-0.5
333	1	435	-0.5
334	1	436	-0.5
335	1	437	-1
301	2	438	-1.5
302	5	401	-0.5
303	7	402	-1
304	10	403	-3.0
305	13	404	-3.0
306	10	405	-5.0
Manifold B	70gpm total	406	-3.0
307	6	407	-7.0
308	6	408	-1.0
309	6	409	-8
310	7	410	-7
311	5	411	-6
312	5	412	-1
313	8	413	-5
314	8	414	-3
315	3	415	-8
316	4	416	-7
317	5	417	-5
318	7	418	-15
Manifold C	95gpm total	419	-8
319	13	420	-6
320	12	421	-22
321	17	422	-16
322	12	423	0.5
323	16	424	-26
324	9	425	-13
325	6	426	-2
326	3	427	-2
327	2	428	-19
328	4	429	-4
329	1	430	-5
		431	-1

Figure 4.2. For comparison, the observed average head differentials for pre-barrier (Feb-Mar 78) conditions are 3.5, 4, and 4.5 ft. Graphical comparison of the distributions is shown on Figure 4.3.

Figure 4.3 clearly shows the influence of the natural pre-barrier potentiometric patterns on the success of barrier operations. The Feb-March 78 differentials were observed for a period of time where the total flux across the barrier alignment is estimated to be 255gpm, some 35gpm greater than the present (May 1987) flux. Despite the difference in interception rate, the distribution of gradient is very similar. Some reduction in differential is realized for the present day equilibrium conditions. It is felt that the slight reduction is due to the influence of the slurry wall and concentration of distributed flows to the point discharge and recharge wells.

Figure 4.4 depicts the cumulative discharge of the pumping wells versus cumulative recharge, plotted by station along the slurry wall barrier for the May 1987 interception flow rates. The slope of these lines at a point, correspond to the flow rate per lineal foot of barrier required to maintain natural equilibrium at that point along it's alignment. The steepest slopes correspond to areas along the barrier which intercept the most water, while the flatest slopes correspond to areas which receive smaller amounts of flow.

The distribution of flow along the alignment varies from less than 0.5 gpm per 100 lineal feet at the eastern and western boundaries, to nearly 20 gpm per 100 lineal feet at the western side of manifold B. The graph indicates an average interception of approximately 3 gpm per 100 lineal feet.

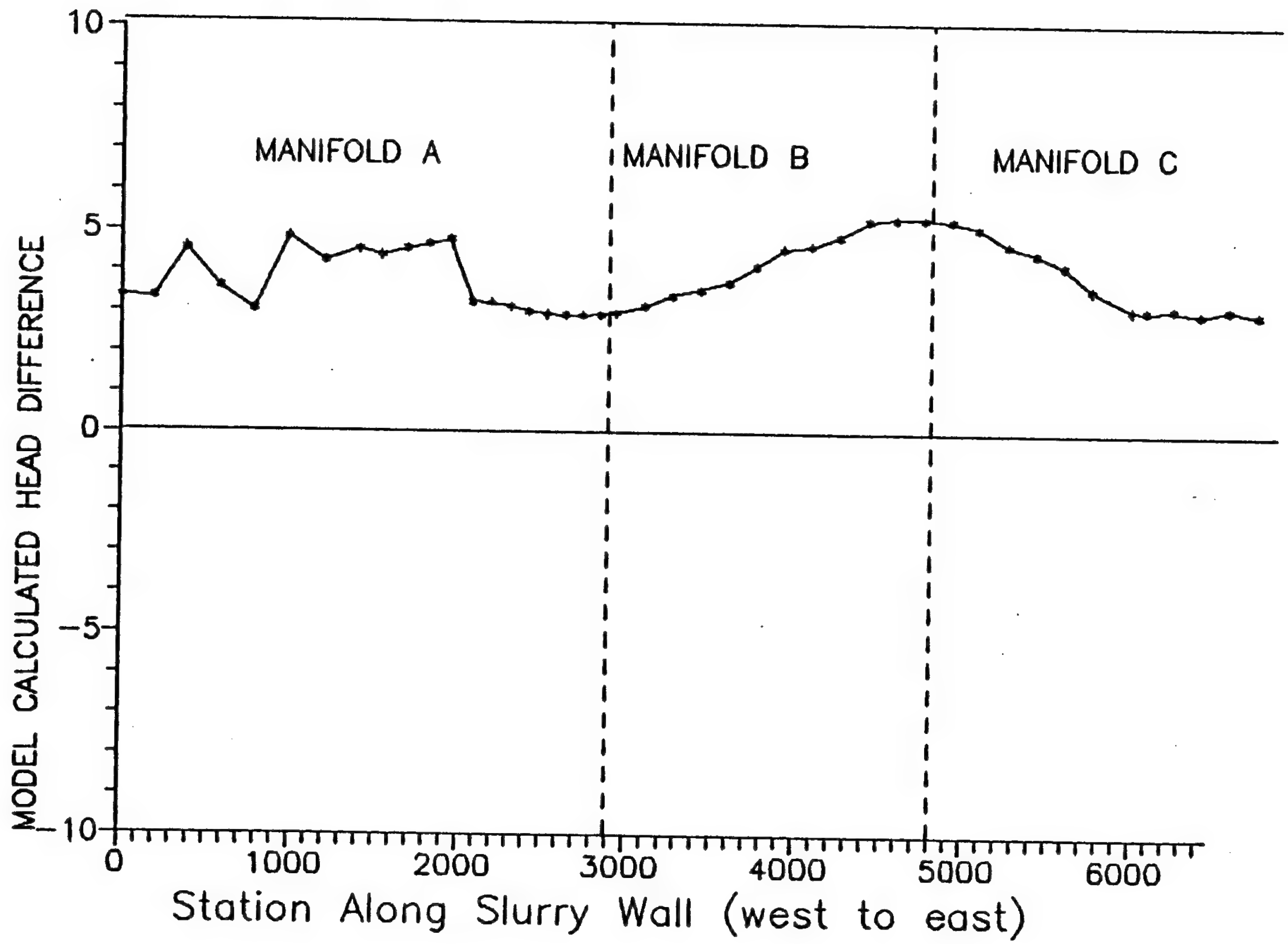


Figure 4.2 Model Computed Head Differential Across the Slurry Wall for Wells Pumping at May 1987 Natural Interception Rates (Table 4.1), Simulation 4.1.a

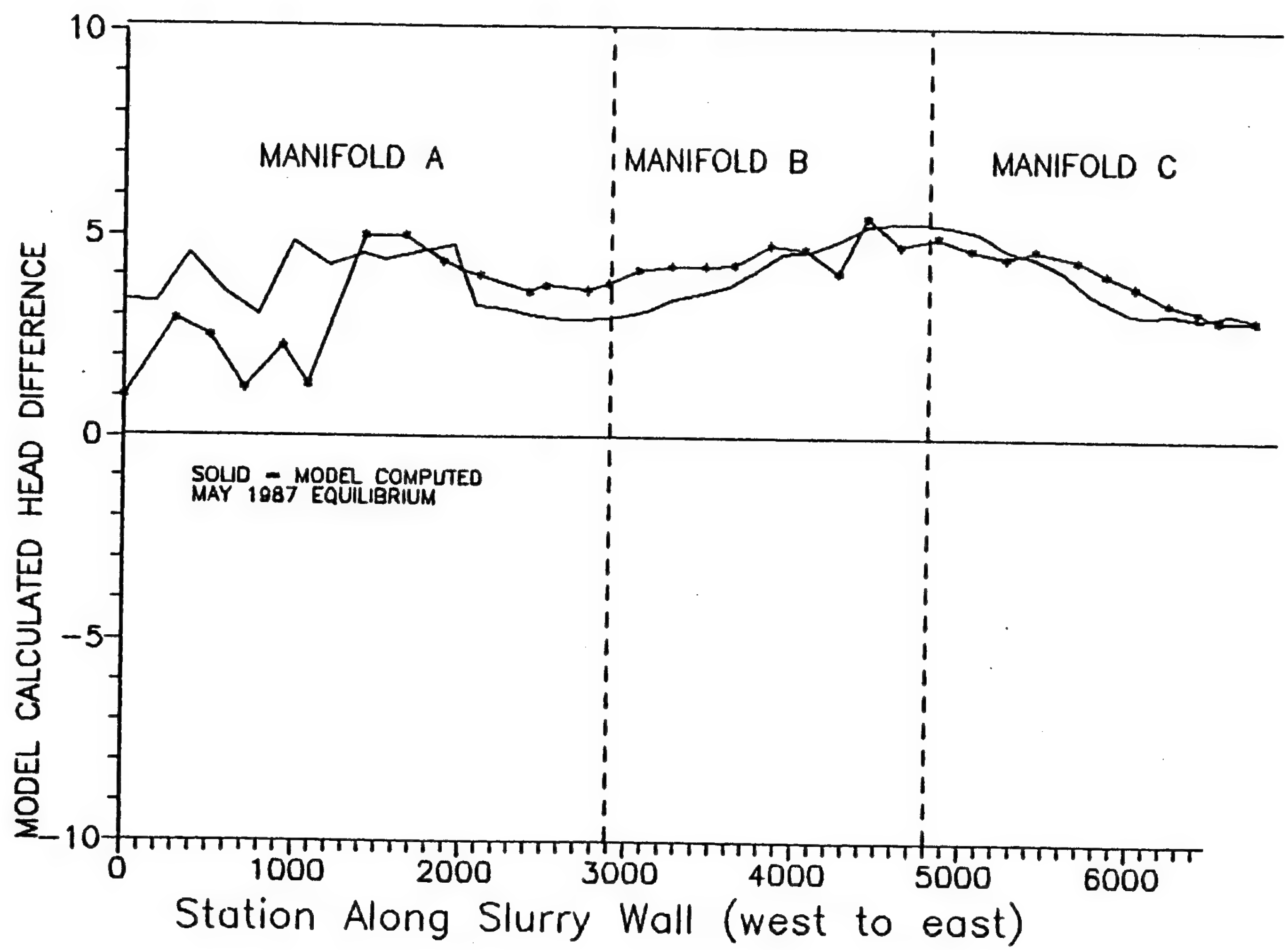


Figure 4.3 Comparison of Pre-barrier Observed and May 1987 Model Estimated Slurry Wall Head Differential Distributions

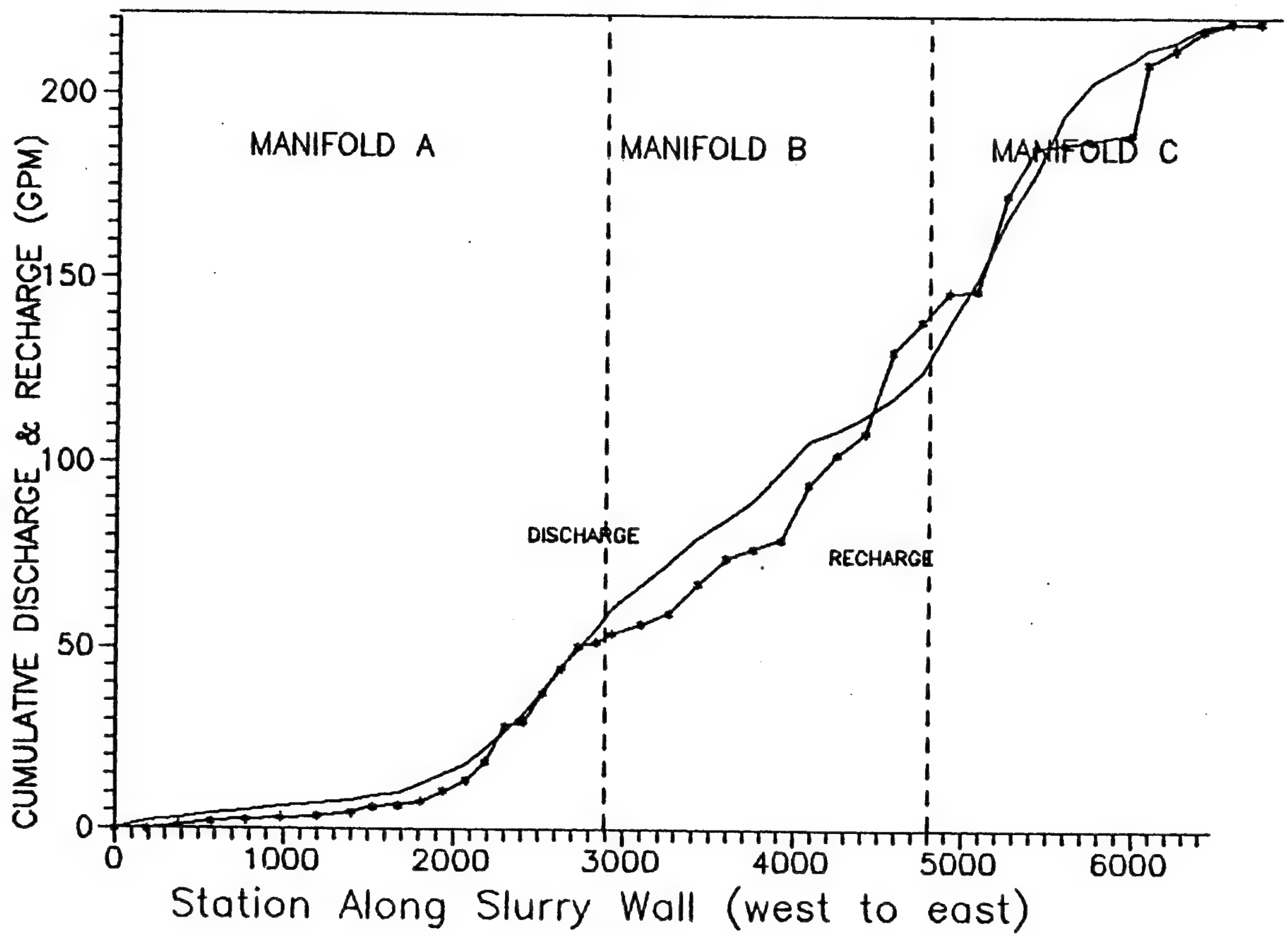


Figure 4.4 Cumulative Discharge and Recharge vs. Distance Along the Slurry Wall

Note that under the natural equilibrium conditions, there is a good agreement between the discharge and recharge sides. It is assumed that this difference between discharge and recharge is due to the fact that the slurry wall and well alignments are not perfectly orthogonal to the principal direction of flow, and some redistribution of flow occurs between the discharge and recharge well alignments under natural conditions.

4.1.b Natural Interception Historical Proportioning

For this simulation, the natural flow intercepted by manifolds A, B, and C are allocated to the individual wells based upon the model computed proportion factors estimated during transient calibration and verification. This simulation represents the theoretical average historical operation of the barrier system assuming that the system would tend toward the natural interception rates on the average.

The distribution of pumping and recharge is shown on Table 4.2. The discharge for well 303, wells, 307 to 312, wells 320 and 328 were set equal to zero. The rates given in Table 4.2 are based upon the historical pump proportioning estimated in transient calibration and verification. Adsorber flow rates are manually controlled by one valve located between the influent wetwells and the adsorbers. Individual average pump rates are controlled by floats located in each well and as a whole manifold by influent wetwell float switches. Wells in one manifold are thought to be pumped preferentially according to their relative capability to produce water. Wells that have the capability to pump continuously are over-pumped,

Table 4.2 Rates- Natural Manifold Interception with Historical Proportioning

Discharge Wells		Recharge Wells	
Number	Rate (gpm)	Number	Rate (gpm)
Manifold A	55gpm total	Total Recharge	-220
330	2	432	-1.5
331	1	433	-1.0
332	2	434	-0.0
333	.5	435	-0.0
334	.5	436	-1.0
335	.5	437	-0.5
301	1.5	438	-0.5
302	2	401	-0.0
303	0	402	-1.0
304	16	403	-1.5
305	11	404	-1.5
306	18	405	-1.0
Manifold B	70gpm total	406	-0.5
307	0	407	-0.5
308	0	408	-2.5
309	0	409	-2.5
310	0	410	-4.5
311	0	411	-3.0
312	0	412	-1.0
313	16	413	-2.5
314	17	414	-2.0
315	7	415	-1.5
316	10	416	-6.5
317	7	417	-16.0
318	13	418	-16.0
Manifold C	95gpm total	419	-16.0
319	8	420	-23.5
320	0	421	-23.5
321	18	422	-10.0
322	18	423	-10.0
323	22.5	424	-6.5
324	5	425	-17.5
325	11	426	-10.0
326	10	427	-15.5
327	2	428	-3.5
328	0	429	-2.5
329	0.5	430	-6.5
		431	-6.5

while wells that may cycle on and off due to inability to produce water equivalent to the pumps capability are under-stressed if total pumpage is limited by the influent sump float switches. The rates given in Table 4.2 may also reflect the effect of discharge from Denver formation to the alluvium.

The model estimated average head differentials across the slurry wall are summarized in Table 4.3. Table 4.3 includes the estimated average head differentials for the natural interception conditions for comparison.

Table 4.3 Comparison of Estimated Slurry Wall Head Differentials for Historical Distribution and Natural Distribution Simulations

Manifold	Historical Distribution	Natural Distribution
A	5.0 ft.	2.5ft.
B	3.0 ft.	4 ft.
C	0.5 ft.	4 ft.

Figure 4.5 depicts the distribution of head differential along the slurry wall alignment. It should be noted that the maximum differential is located at the 1800-2000 foot distance along the slurry wall. This point roughly corresponds to the location of a narrow paleochannel. Another narrow paleochannel is located at approximate the 6,000 ft. distance. However, historical recharging to the bog minimizes the head differential in this area.

Results of the historical proportioning simulation indicate a more lop-sided distribution of head differential across the slurry wall then that computed for natural interception rates. The greatest head differential is observed over Manifold A which includes the original pilot scale system.

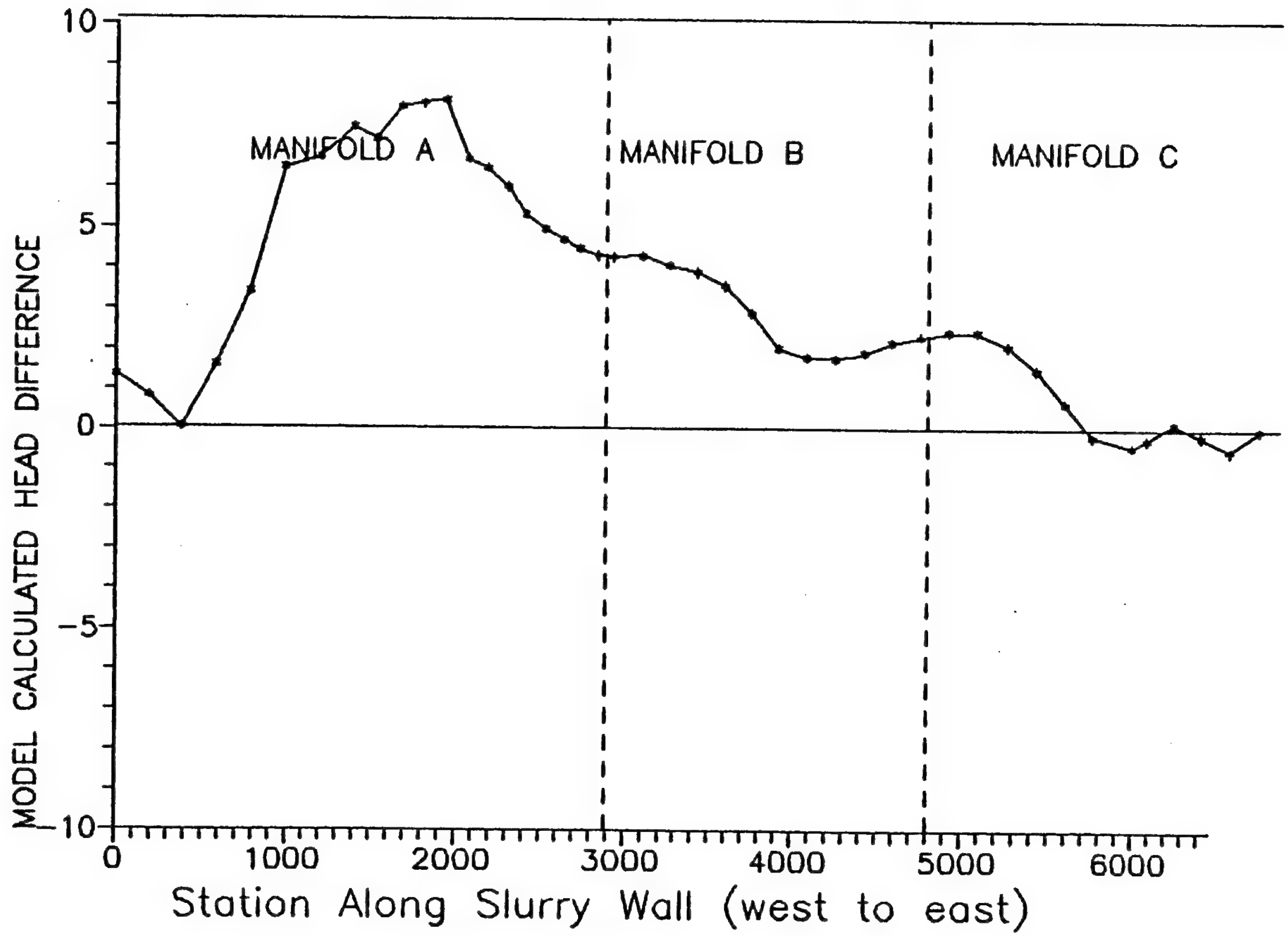


Figure 4.5 Model Computed Head Differential for Present Natural Manifold Interception, Distributed Based Upon Historical Proportioning

This differential is much less desirable considering the fact that Manifold A intercepts the water with the highest contaminant concentrations. It is thought that the historical pumping distribution is the result of the inability to monitor and control individual pump and recharge rates due to the mechanics of the system. Production capability of individual wells varies along the barrier, presumably due to its location relative to the incised paleochannels and areas where the alluvium is cemented. Individual wells on each manifold are over-pumped while others are under-pumped based upon relative capability to produce water. Wells that are over-pumped, should be valved down, to allow a more equitable distribution of differential head across a manifold. To complicate matters further, on the recharge side of the barrier all the recharge wells are connected to a common manifold. Consequently, the quantity of recharge across the slurry wall does not mirror what is being pumped out on the upgradient side. The recharge distribution lateral is pressurized, it is possible that pressure distribution in the lateral effects the performance of individual wells. The distribution of recharge may change with time, as wells clog with carbon fines. Differential clogging of recharge wells has been observed by barrier operations personnel. Areas along the barrier with the greatest capability for recharge at the given time are preferentially recharged while other areas receive negligible recharge. Furthermore, operations personnel have indicated that water that cannot be recharged through non-productive or clogged wells has been discharged to the bog. Consequently, this area has been over recharged in the past, causing non-equitable distributions of recharge and discharge across the barrier.

Table 4.4 presents a comparison of pumping and recharge across the barrier by manifold. Figures 4.6 and 4.7 depict the comparison of cumulative

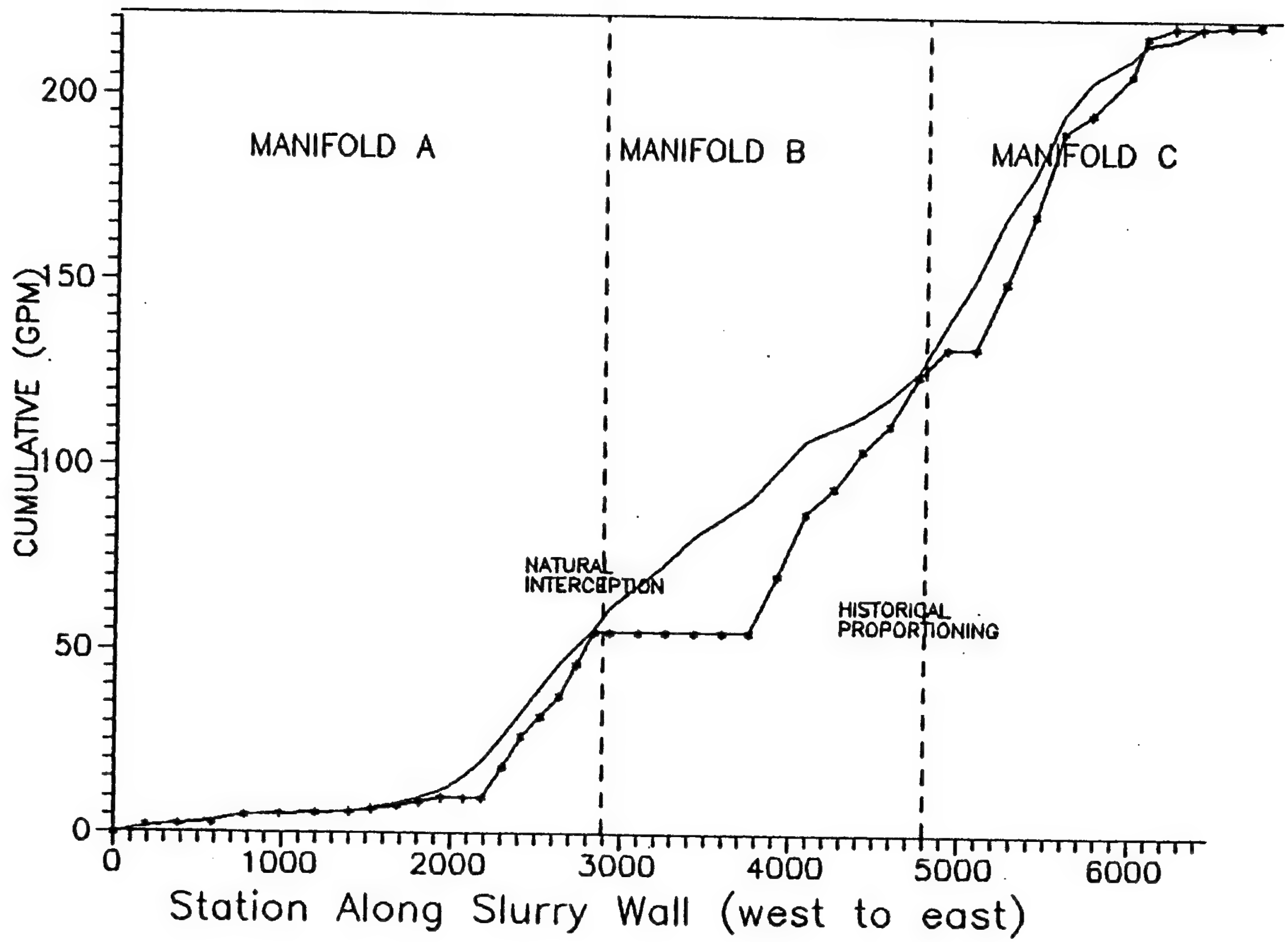


Figure 4.6 Comparison of Cumulative Discharge for Natural Interception and Historical Proportioning

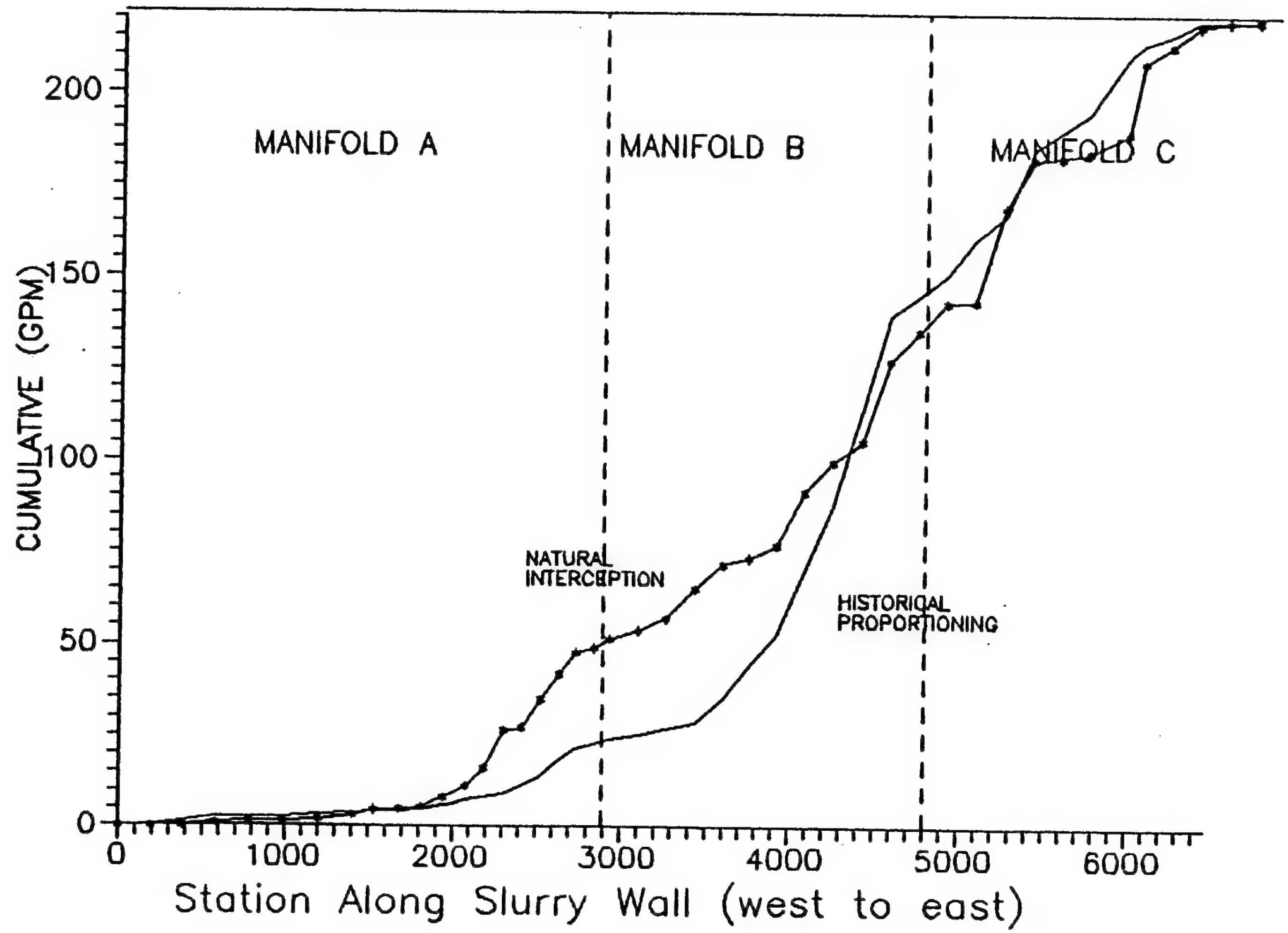


Figure 4.7 Comparison of Cumulative Recharge for Natural Interception and Historical Proportioning

pumpage and recharge for the historical and natural interception distributions. The solid line corresponds to natural interception, while the points correspond to the historical distributions. Review of figure 4.6 indicates that despite the natural equilibrium flow to the manifolds being pumped as a whole, the distribution of pumping varies considerably.

Table 4.4 Comparison of Pumping and Recharge by Manifold, Historical Proportioning

Manifold	Sum of Discharge Wells	Sum of Opposing Recharge Wells
A	55	23
B	70	117
C	95	80

Figure 4.6 indicates that manifold A is being pumped at equilibrium or slightly underpumped. Figure 4.7 indicates that manifold A is recharged at equilibrium for the first 2,000 feet of its alignment and then is under recharged for its remaining length. Manifold B is largely underpumped and under recharged for the first 1,000 feet of its alignment and then is overpumped and over recharged over its remaining length. Figure 4.7 indicates that this is particularly true for recharge where the slope of the cumulative recharge line is much greater than equilibrium from station 3800

to station 4800. The reason for this is the excessive recharge to the bog in the past. The net result for manifold B is that greater than equilibrium recharge is attained and near equilibrium pumping is occurring. However, in neither case is there an equitable distribution between discharge and recharge. Manifold C is receiving the natural interception rate, but the distribution of recharge favors the west end of manifold C which is over an ancestral channel of first creek. The eastern end of manifold C is pumped and recharged at less than equilibrium.

Figure 4.8 Depicts the relationship between cumulative pumping and recharge across the manifold for the historical proportioning. A comparison of slopes indicates that between stations 2000 and 5500, recharge does not ever mirror discharge as compared with the natural interception conditions. Furthermore, excessive recharge to the bog has caused the distribution of discharge and recharge to be grossly non-equitable.

4.1.c Present Operating Conditions

This simulation consists of modeling the total barrier interception flux of 220 gpm allocated to the manifolds according to proportioning observed for the months of January through April 1987. Currently, manifold C is overpumped and manifold A and B are underpumped

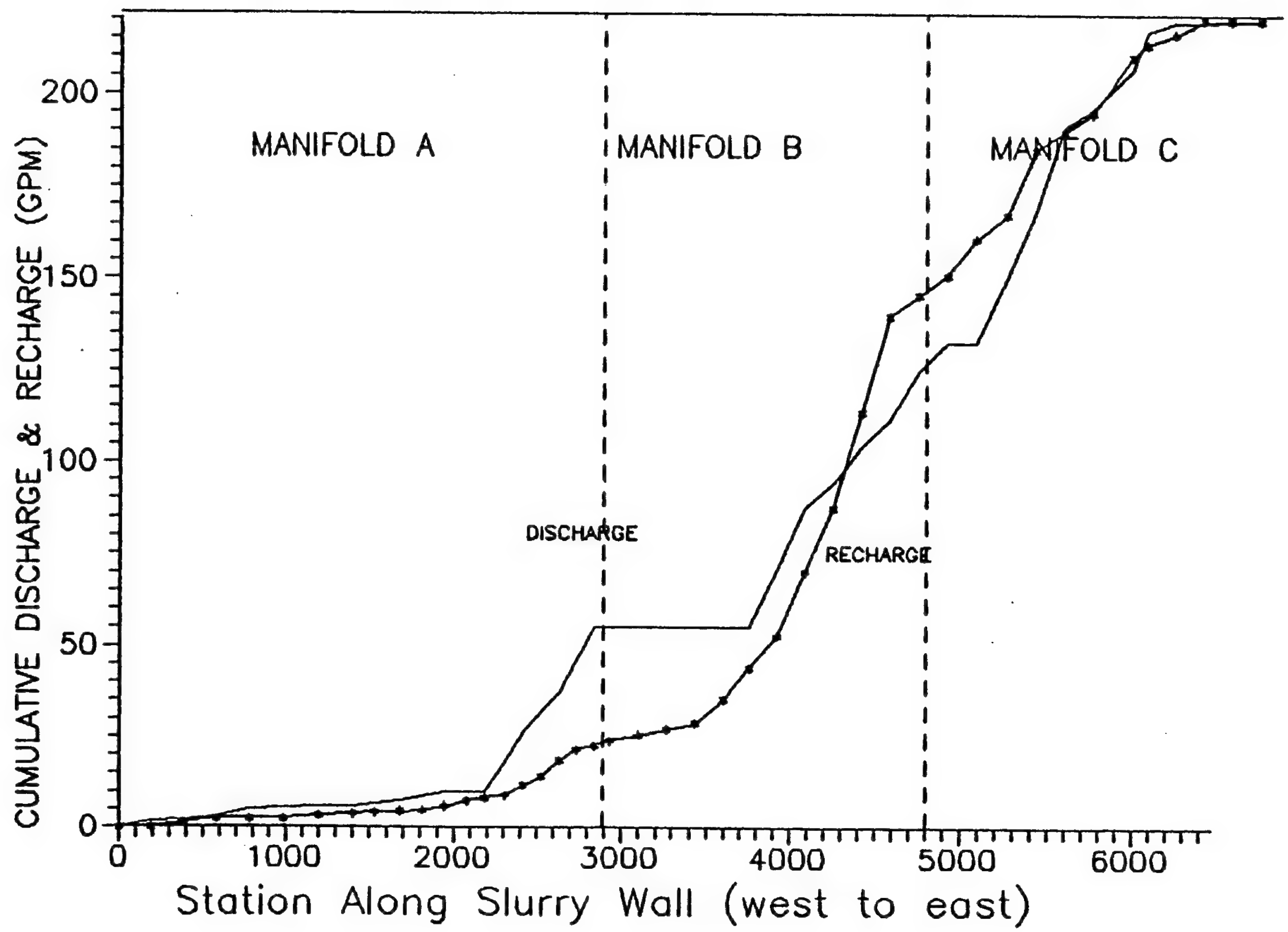


Figure 4.8 Cumulative Discharge Pumping vs. Cumulative Recharge Along the Slurry Wall Alignment for Historical Proportioning, Simulation 4.1.a.

Proportioning the May 1987 interception flux rate of 220 gpm to manifolds A through C according to the proportions observed for the four month period resulted in the manifold rates shown in Table 4.5. The manifold flow rates were then distributed to each individual well according to the historical proportioning. The individual well rates are listed in Table 4.6. The well rates can be determined by multiplication of the manifold total by the individual well proportion factor listed in Chapter 3.

Table 4.5 Manifold Pump Rates For Simulation 4.1.c

Manifold	Total Discharge	Opposing Recharge
A	40gpm	23gpm
B	70gpm	117gpm
C	108gpm	80gpm
Totals	220gpm	-220gpm

Figure 4.9 depicts the estimated head differential across the bentonite slurry wall for the current operating conditions outlined above. Table 4.7 lists the average head differential for this scenario and the natural interception scenario. Comparison of average differentials for this scheme with those of natural interception indicate that this pumping and recharge scenario is largely different than equilibrium. As with simulation 4.1.b Figure 4.9 indicates that manifold C overpumping produces a negative head differential near the eastern end of manifold C where the aquifer is then, and a positive differential over the First Creek paleochannel. However,

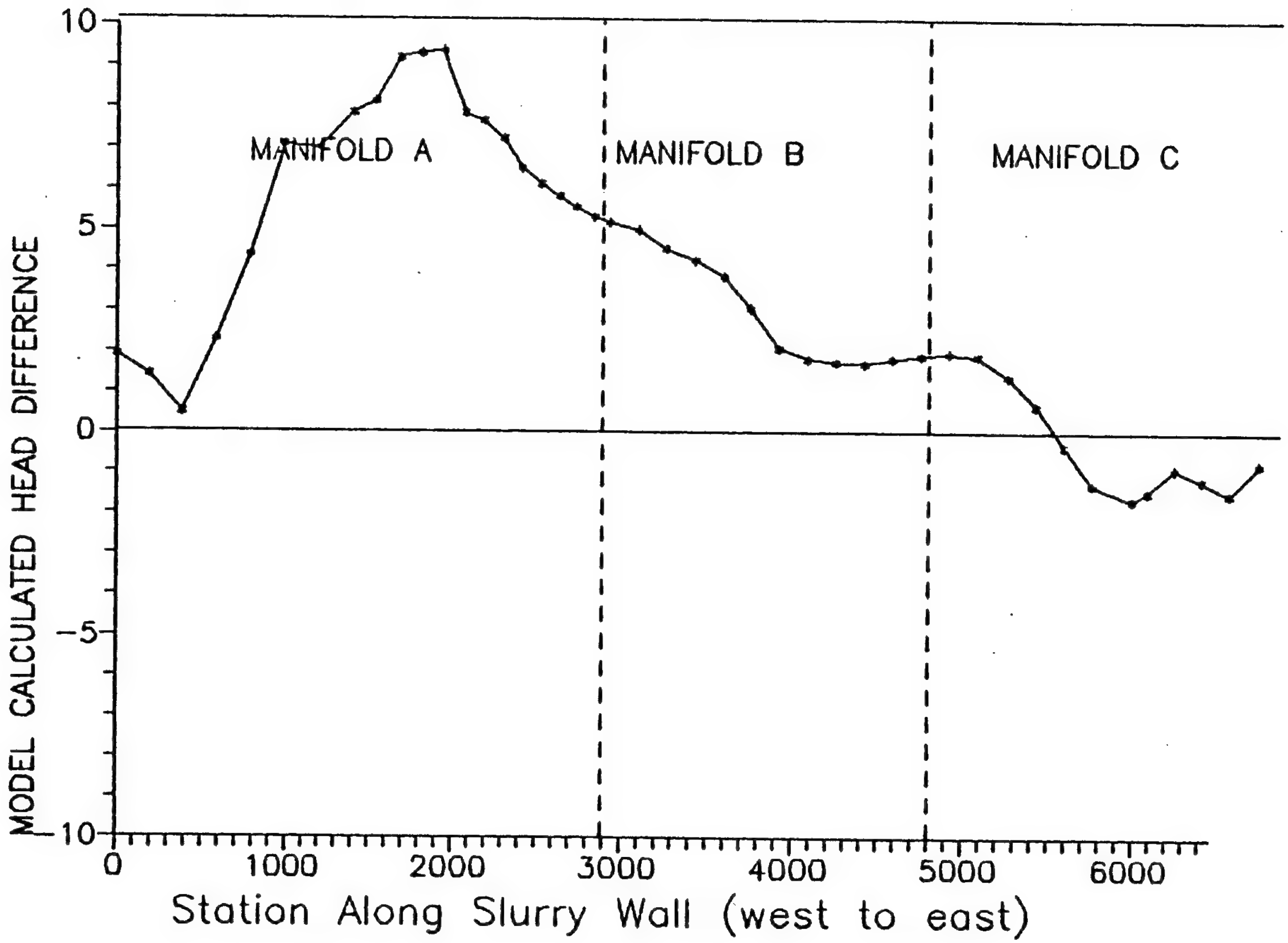


Figure 4.9 Model Estimated Head Differential, Present Operating Conditions

Table 4.6 Rates - Present Operating Conditions with Historical Proportioning

Discharge Wells		Recharge Wells	
Number	Rate (gpm)	Number	Rate (gpm)
Manifold A	40gpm total	Total Recharge	-220
330	1.5	432	1.5
331	0.5	433	1.0
332	1.5	434	0
333	0.5	435	0
334	0.5	436	1.0
335	0.5	437	0.5
301	1.0	438	0.5
302	1.5	401	0.5
303	0	402	0.5
304	12.0	403	1.0
305	8.0	404	1.5
306	12.5	405	1.0
Manifold B	72gpm total	406	0.5
307	0	407	0.5
308	0	408	2.5
309	0	409	2.5
310	0	410	4.5
311	0	411	3.0
312	0	412	1.0
313	16.0	413	2.5
314	17.5	414	2.0
315	7.0	415	1.5
316	10.5	416	5.5
317	7.0	417	*
318	14.0	418	*
Manifold C	108gpm total	419	*
319	8.5	420	*
320	0	421	*
321	20.0	422	9.0
322	21.0	423	8.0
323	26.0	424	6.5
324	5.5	425	15.5
325	12.0	426	8.0
326	11.5	427	15.0
327	3.0	428	3.5
328	0	429	2.5
329	0.5	430	6.5
		431	6.5

The bog was modeled as constant head in this scenario. Total recharge = 105gpm

overpumping manifold C causes the average differential to be reduced only three quarters of a foot from simulation 4.1.b. This demonstrates the inelasticity of the head differential in this region to increases in pumping. A change in total manifold rate of 13 gpm resulted in very little changes in differential head across the barrier. Manifold A under went a change in average differential of one foot over simulation 4.1.b. This is a more substantial change than that observed for manifold C since nearly the same amount of change in pumpage is occurring over a larger region. However, even this change in head differential is not as large as some might expect, indicating that the high head differential is largely caused by the under-recharging in this area.

Table 4.7 Comparison of Present Pumping versus Natural Interception
Average Manifold Head Differential

Manifold	Present Pumping	Natural Interception
A	6.0 .	2.5ft.
B	3.0	4ft.
C	-0.25 .	4ft.

Figure 4.10 presents comparison of cumulative discharge for the natural interception and present operating simulations. Figure 4.11 depicts the relationship between cumulative discharge and recharge versus station along

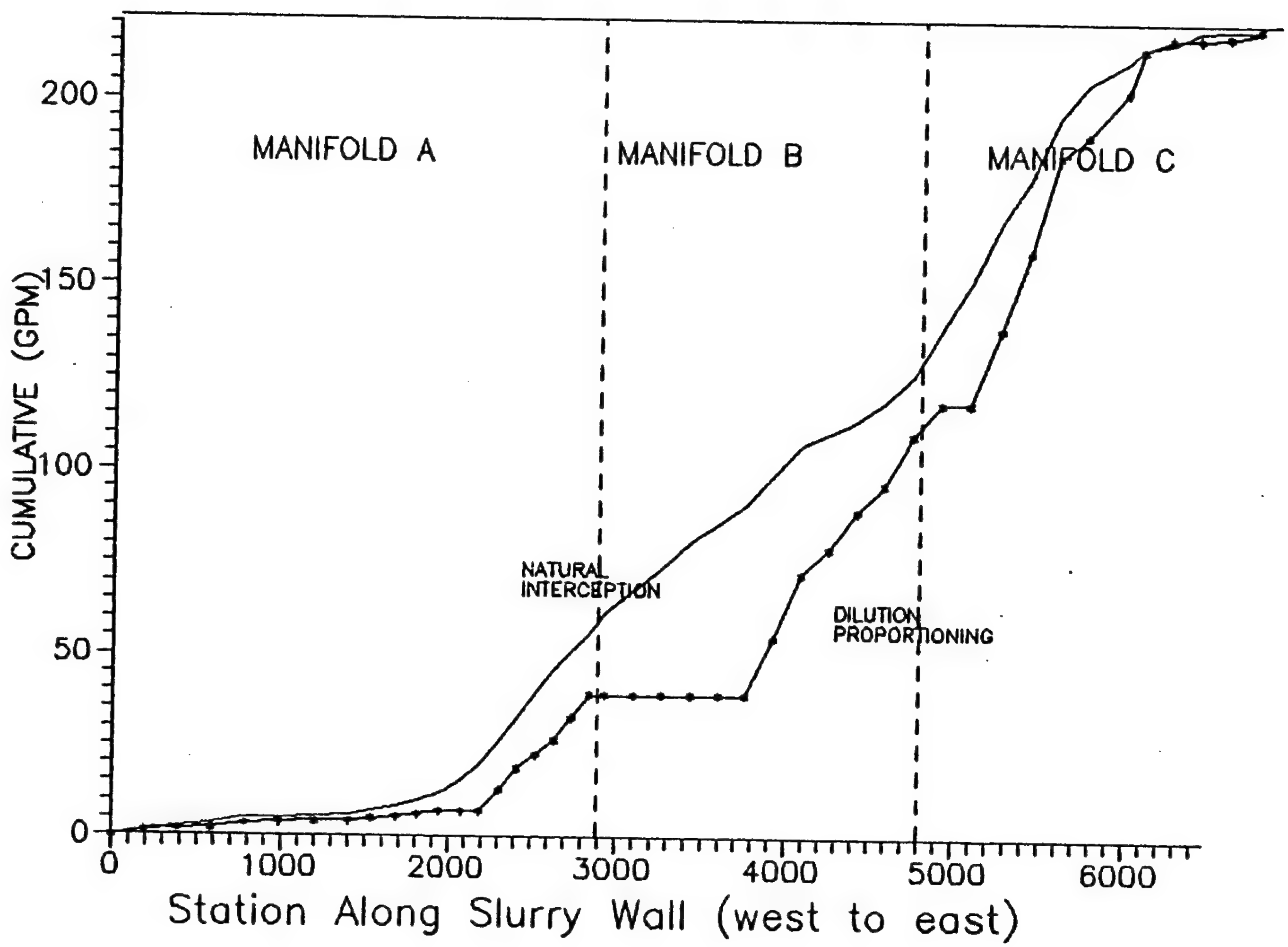


Figure 4.10 Comparison of Cumulative Discharge for Natural Interception and Present Operating Simulations

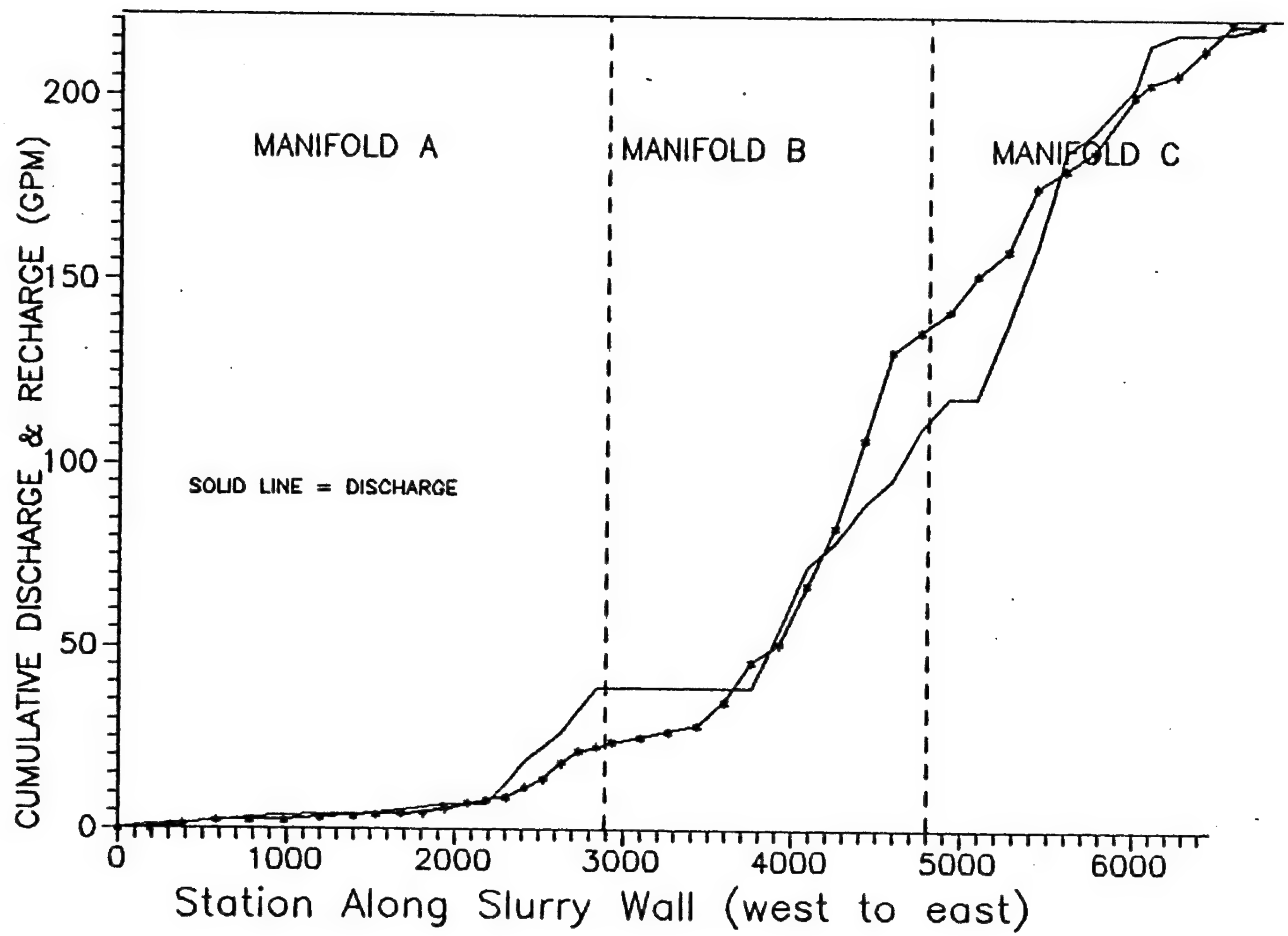


Figure 4.11 Cumulative Discharge and Recharge vs. Station Along Slurry Wall

the barrier alignment. Review of Figure 4.10 indicates a greater disparity between the manifold C overpumping scheme and natural interception. As could be expected, the entire length of manifold A is being under-pumped. Under-pumping of manifold B wells is more dramatic than for the historical distribution simulation. Much of manifold C is over-pumped, resulting in the reduction in differential from the previous simulation.

4.2 Barrier Reconfiguration Simulations

It is apparent based upon the simulations presented in Section 4.1 that a reverse gradient across the entire length of the slurry wall is likely not possible for the current barrier system configuration. The natural pre-barrier gradient in the vicinity of the system largely limits the ability to approach or attain a reversal. Operation of the system at the natural interception rates yields the lowest head differential for the region of barrier that receives the most highly contaminated water. The average differential for this pumping scheme is indicated to be on the order of two and one half (2.5) feet. Over manifold A the average differential estimated for the historical operation of the barrier system is indicated to be nearly five (5) feet.

Many barrier reconfiguration simulations were performed in order to evaluate what improvements in head differential could be realized with a minimum of re-design of the system. It soon became obvious that if a reverse gradient was desired over all or at least part of the barrier alignment, major reconfiguration of the discharge and recharge wells was needed. For some of the simulations, it was assumed that the system could

be modified such that individual pump rates could be selected to match the natural interception flow rates computed for the respective well location.

The simulations presented in the sections to follow represent reconfiguration schemes that resulted in improvement in the head differential across the slurry wall barrier on the order of one to two feet. It is felt that such a reduction would be required to justify the effort and expense in modification of the system. It appears based upon the simulation results that a major reconfiguration would be required to make a significant change in the potentiometric conditions at the slurry wall.

4.2.a Zero Head Differential Simulation

Although it appears that a reverse gradient is not likely with the present configuration, it was necessary to determine the hypothetical conditions under which a reverse gradient or at least a zero gradient across the slurry wall might be attained under the present configuration. A steady state simulation was performed at which the heads at the present recharge wells were set to a constant value equal to heads at the discharge wells that would be seen at the end of the run. The result was near zero head differential. Total manifold rates were at natural equilibrium while the discharge wells were pumped at the historical distribution, thus representing the theoretical average pumping rate assuming the system would tend toward natural conditions. The results of this simulation indicated that a total of 290 gpm would be required as recharge to attain a zero head differential along the entire length of the slurry wall while maintaining the pumping wells at 220 gpm. This is strictly a hypothetical flux rate since the natural flux to the barrier is only 220 gpm. However, the results make it apparent that if a reverse gradient is desired over all or at least

part of the barrier alignment, major reconfiguration of the discharge and/or recharge wells is needed. To obtain the 290 gpm for recharge on a long term basis would require import of additional water to the barrier.

4.2.b Zero Head Differential Along the Barrier Alignment with all Recharge Wells Moved to 45 feet from the barrier

This simulation is very similar to 4.2.a except that all the recharge wells were moved to the line of nodes 45 feet away from the barrier. The results of this simulation indicate that a total of 233 gpm is needed as recharge to attain a zero gradient along the entire length of the slurry wall. This amount is considerably less than that computed with the wells at the present configuration. However, the total is still 13 gpm over the natural flux. Figure 4.12 illustrates the comparison of the cumulative fluxes for natural recharge conditions and fluxed calculated in this simulation assuming that the natural fluxes to the wells at the 45 foot line are the same as those for the present configuration. The figure indicates that the difference in cumulative pumpage builds over the eastern part of manifold A. However, for the remaining part of the barrier, recharge appears to mirror natural recharge. This indicates that the best and most desirable distribution of head differential across the slurry wall will result if recharge occurs at near natural equilibrium fluxes.

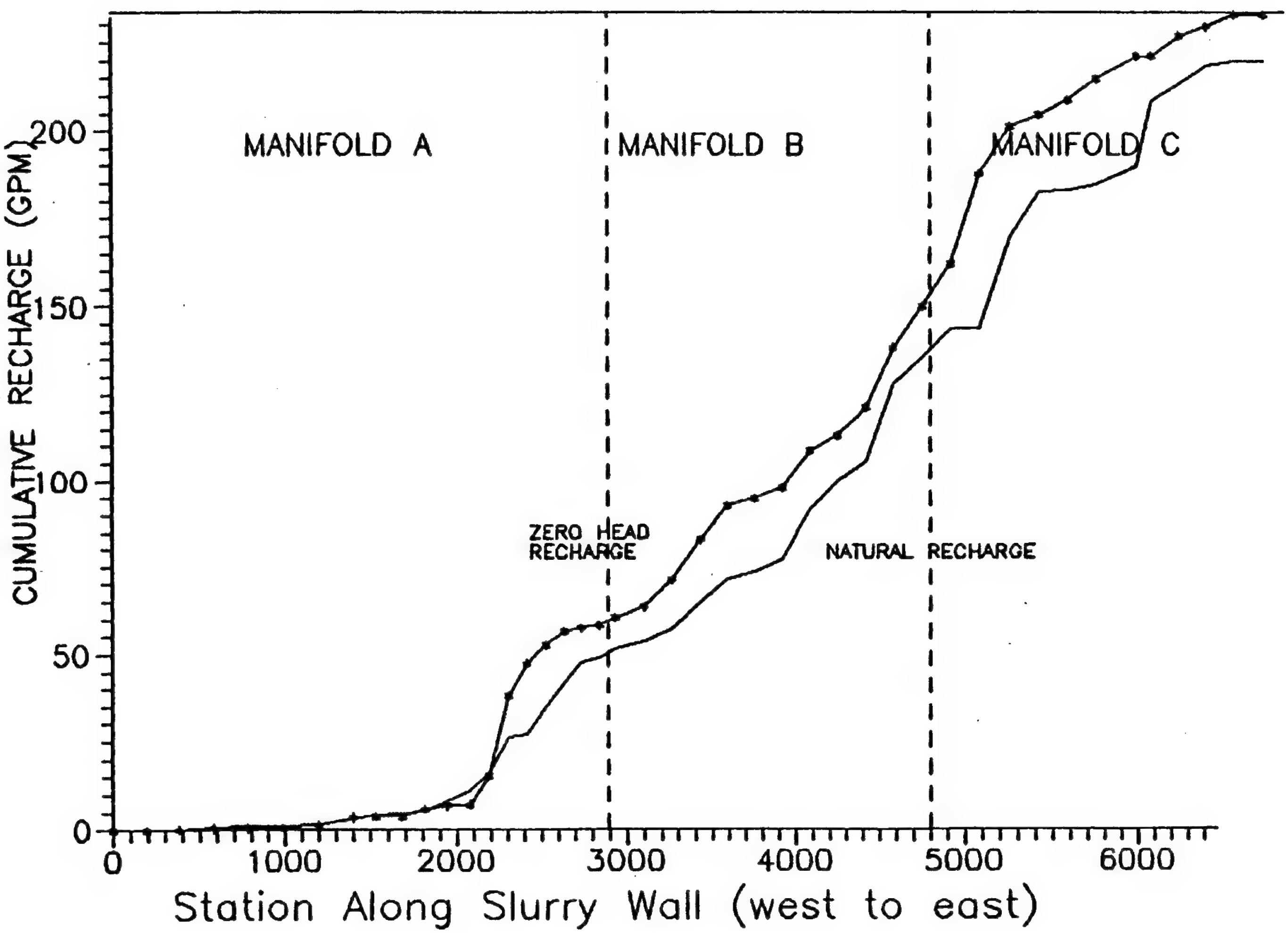


Figure 4.12 Comparison Recharge Fluxes for Natural Equilibrium and Simulation 4.2.b

4.2.c.1 Relocation of all recharge wells to 45 feet, Pumping at Historical Proportions and Present Manifold Rates

To compare with simulation 4.2.b, a simulation was performed in which recharge wells were moved to the 45 foot line and recharged at historical proportioning. Similarly discharge wells were pumped at historical proportioning. The rates are shown in table 4.6. Figure 4.13 illustrates the head differential resulting from this simulation. As can be observed, a negative head differential has resulted over much of the barrier. However, there is still a significant positive gradient over much of the eastern part of manifold A and the western part of manifold B. The region includes most of the pilot scale system where the most highly contaminated water is intercepted. The average head differential for manifolds A, B and C are 0.4 feet, 0.3 feet and 3.0 feet respectively. It should be noted that this simulation is more realistic than 4.2.b in that a total flux of 220 gpm is recharged.

4.2.c.2 Relocation of all Recharge Wells to 45 Feet, Pumping and Recharging at Natural Equilibrium Flux Rates

This simulation is the same as 4.2.c.1 except that pumping and recharge occurs at the natural equilibrium rates listed in table 4.1. Figure 4.14 illustrates the head differential across the barrier resulting from this simulation. A considerable reduction in positive gradient over simulation 4.2.c.1 can be observed near station 2000. Although the average head differentials of 0.4, 0.8, and 0.7 for manifolds A, B, and C do not reflect improved conditions, Figure 4.14 indicates that the head differential along the slurry wall is more equitable than

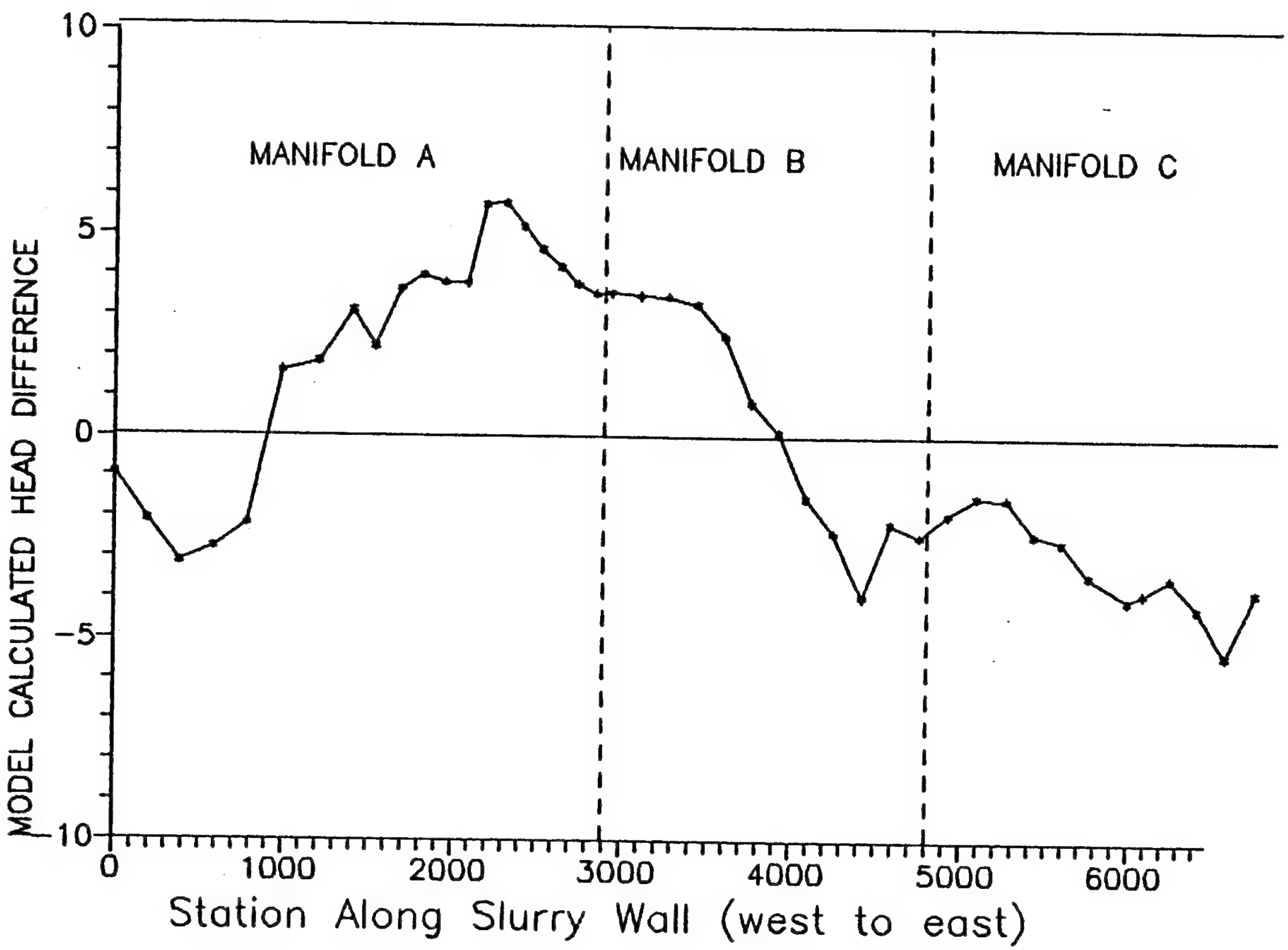


Figure 4.13 Estimated Head Differential Distribution, Simulation 4.2.C.1

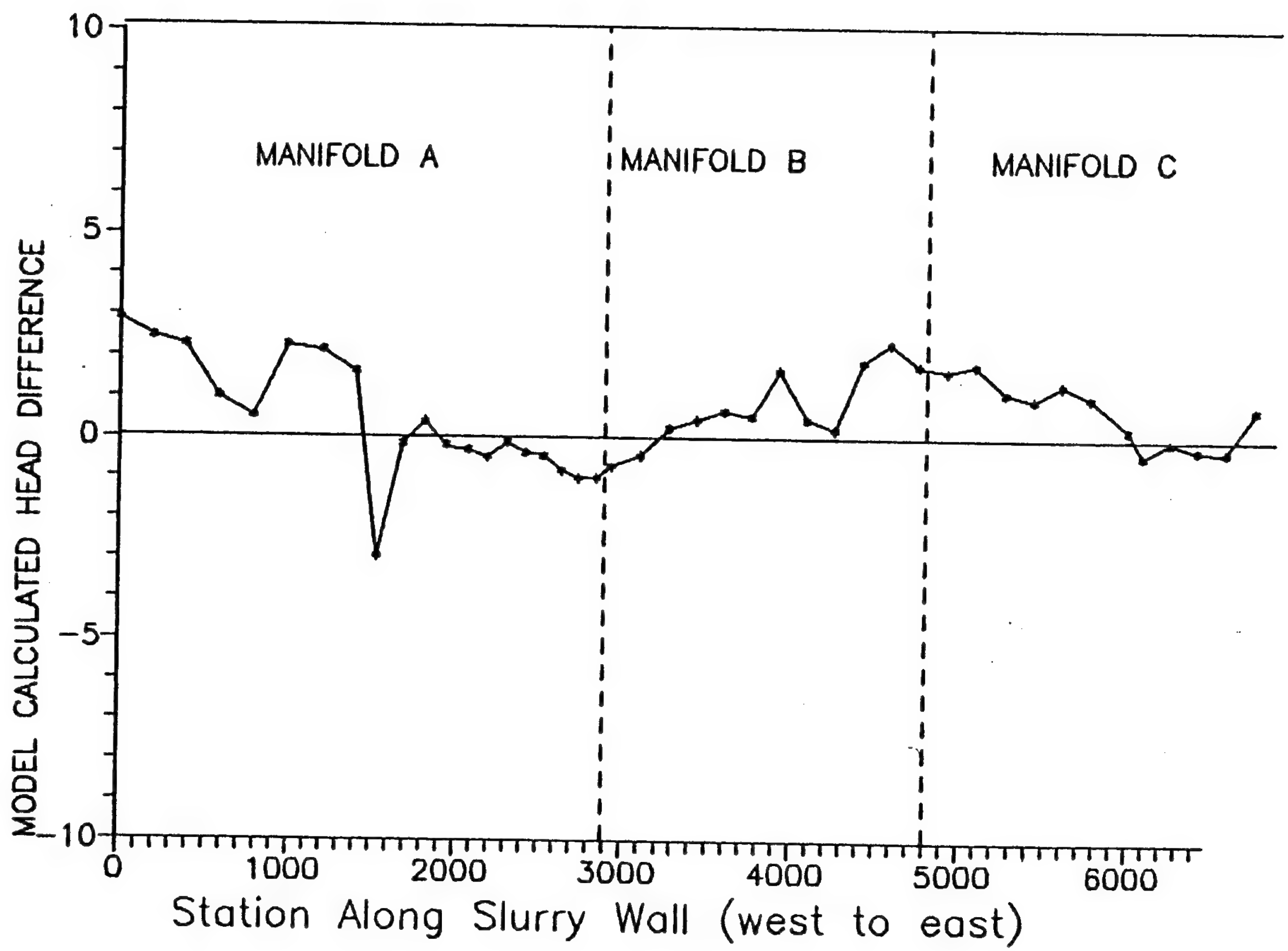


Figure 4.14 Estimated Head Differential Distribution, Simulation 4.2.C.2

simulation 4.2.c.2 reflecting the influence of the natural gradient. As predicted in simulation 4.2.b this comparison indicates that a more desirable distribution of head differential is obtained under natural pumping and recharge conditions .

4.2.d.1 Relocation of all Recharge wells to 45 feet from the Barrier and all Dewatering wells to 150 feet from the Barrier, Present Pumping

This simulation was performed to determine the effect of a major barrier reconfiguration on the head differential across the slurry wall. This simulation involves relocation of all pumping and recharge wells closer to the barrier. The wells were operated at historical proportioning and present pumping as listed in Table 4.6. Figure 4.15 illustrates the head differential across the barrier resulting from this simulation. The average head differentials for manifolds A, B, and C are 1.2 , -1.6, and -4.5 feet respectively. Although these averages indicate a negative differential over most of the barrier, figure 4.15 indicates that a significant positive, gradient still exists over parts of manifold A and B.

4.2.d.2 Relocation of all Recharge and Dewatering Wells to 45 Feet and 150 Feet from the Barrier Respectively, Fluxes at Natural Equilibrium

This simulation is identical to 4.2.d.1 except that pumping and recharge occur at natural equilibrium rates listed in Table 4.1.

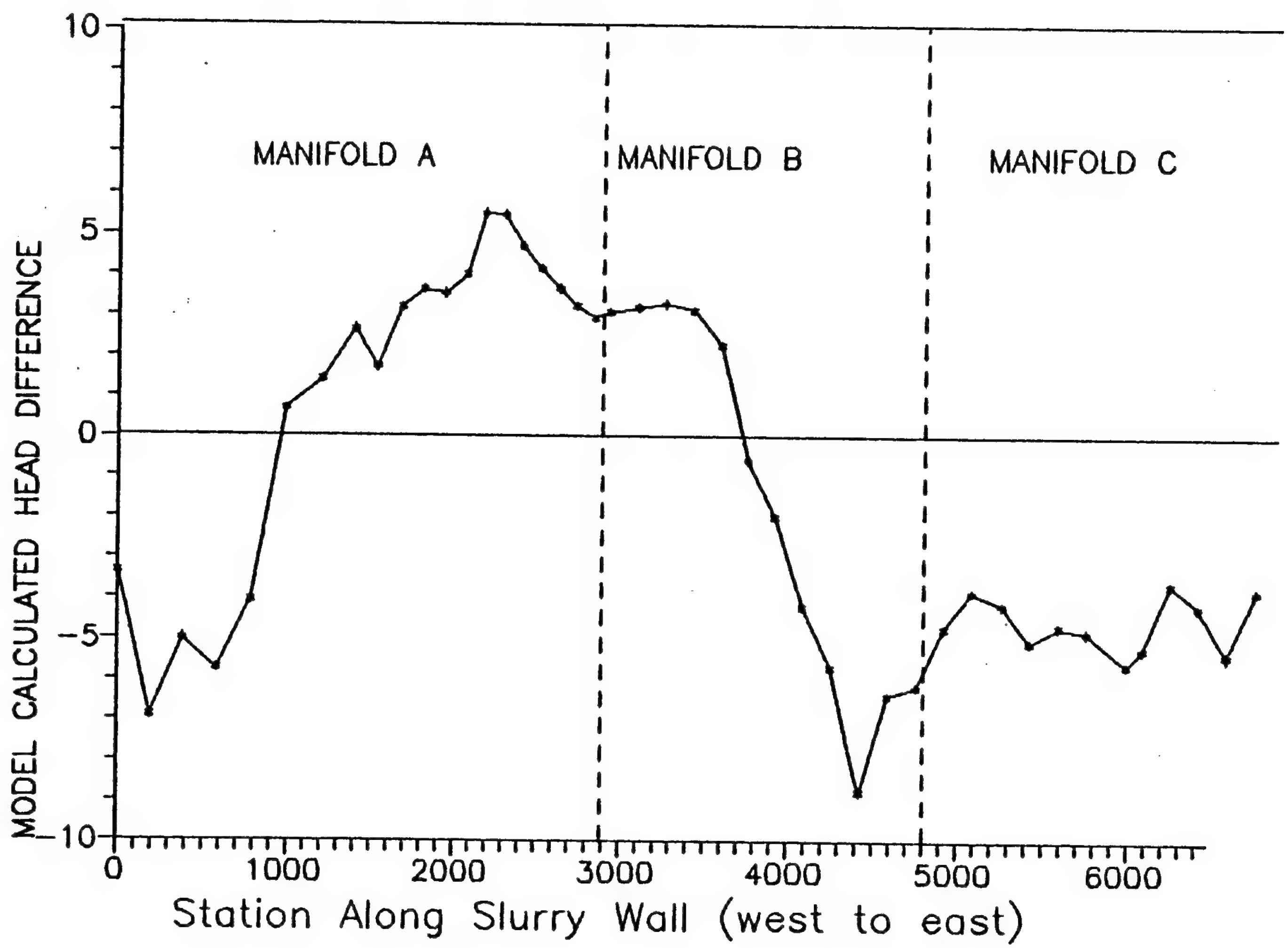


Figure 4.15 Estimated Head Differential Distribution, Simulation 4.2.D.1

Figure 4.16 illustrates the resulting head differential and indicates that except for manifold A, a near zero or reverse gradient is attained. The average head differentials are -0.6, -0.5 and -0.4 for manifolds A, B and C respectively. This scenario represents the most desirable pumping and recharge scheme in that it results in the lowest and most equitable head differential distribution along the entire length of the barrier. The head differential along the slurry wall has been reduced at least an average of 4 feet over all schemes involving the present configuration. Furthermore, the head differential is negative of near zero over much of the barrier. However a positive gradient still exists parts of manifold A and B

Table 4.8 summarizes the model estimated average head differentials across the slurry wall for simulations 4.1.a, 4.1.c, 4.2.c.1, 4.2.c.2, 4.2.d.1 and 4.2.d.2. Analysis of Table 4.8 indicates that the present configuration results in substantial positive gradients across the slurry wall. Furthermore, it indicates that only by reconfiguration of the present system can positive gradients be reduced and a reverse gradient be approached. The manifold C overpumping scenario results in the largest positive gradients over the region of the barrier that intercepts the most highly contaminated groundwater (manifold A). Pumping and recharging at natural equilibrium flux rates results in a more equitable distribution of head differential along the slurry wall and a much lower positive gradient over manifold A. Simulation 4.2.d.2 is the most favorable pumping and recharging scenario in that an average negative head differential results over the entire barrier.

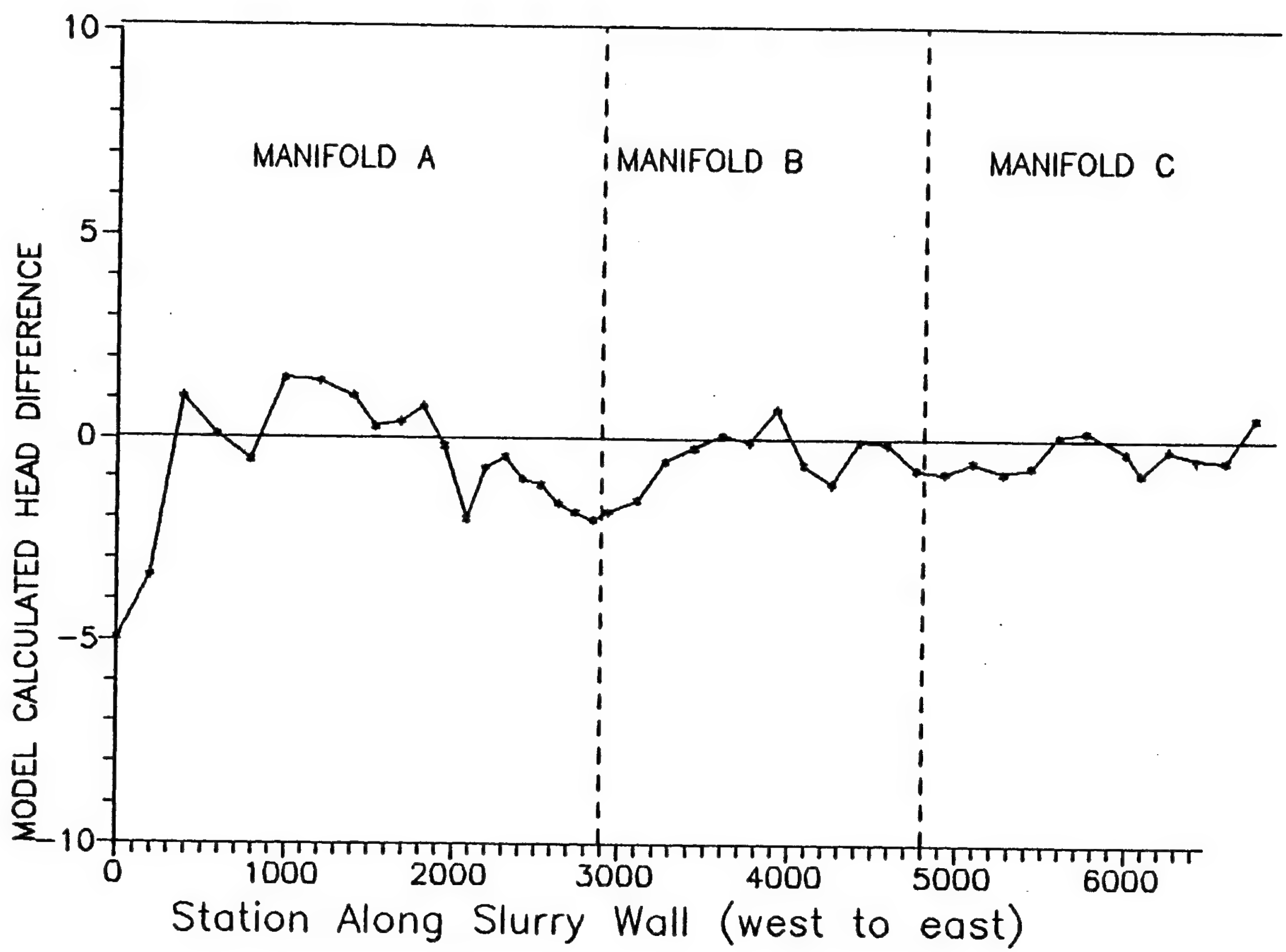


Figure 4.16 Estimated Head Differential Distribution, Simulation 4.2.D.2

Table 4.8 Comparison of Estimated Average Head Differential

Simulation		Manifold A	Manifold B	Manifold C
4.1.a.	(Natural equilibrium)	2.5	4.0	4.0
4.1.c	(Manifold C overpuming)	6.0	3.0	-0.25
4.2.c.1	(Recharge wells at 45 feet, Dilution pumping)	0.4	0.3	-3.0
4.2.c.2	(Recharge wells at 45 feet, Natural equilibrium)	0.4	0.8	0.7
4.2.d.1	(Recharge wells at 45 feet Discharge wells - 150 feet, Dilution pumping)	1.2	-1.6	-4.5
4.2.d.2	(Recharge wells at 45 feet Dicharge wells - 150 feet, Natural equilibrium pumping)	-0.6	-0.5	-0.4

4.2.d.3 Utilization of Existing Pumping System Augmented by Additional Pumping Wells Closer to the Barrier

Historically, the North Barrier System pumping wells have not been pumping at natural interception for the following reasons:

- Some wells are not as well developed as others and are unable to pump the natural interception rate.

- Others wells that are well developed, overpump to compensate for underdeveloped wells.

- Valve mechanics and the location float on-off switches prevent pumping at natural interception.

This simulation was performed to determine if additional wells located closer to the barrier could aid in compensating for wells that pump under natural interception. Table 4.9 is a comparison of historical well discharge rates and natural interception rates. Areas that have been historically overpumped include the east end of manifold B and most of manifold C. Underpumping areas include the west end of manifold B and most of manifold A. This simulation involved lowering the flow rate at overpumped wells to its natural interception flow and keeping the underpumped well flow rates at their present value. Next, a line of constant head nodes was placed between the present wells and the slurry wall. The heads were set to the natural equilibrium head value determined in sumulation 4.1.a. This scheme will allow the model to calculate the flow to the line of constant head nodes needed to maintain natural equilibrium

Table 4.9

Natural vs. Historical Discharge Rates

Discharge Wells

Discharge Well	Natural (gpm)	Historical (gpm)	Comment
Manifold A			
330	2	2.0	
331	2	0.5	UP
332	1	1.5	OP
333	1	0.5	UP
334	1	0	UP
335	1	0	UP
301	2	1.0	UP
302	5	1.5	UP
303	7	0	UP
304	10	11.5	OP
305	13	8.0	OP
306	10	12.5	OP
Manifold B			
307	6	0	UP
308	6	0	UP
309	6	0	UP
310	7	0	UP
311	5	0	UP
312	5	0	UP
313	8	16.0	OP
314	8	17.0	OP
315	3	7.0	OP
316	4	10.0	OP
317	5	7.0	OP
318	7	13.5	OP
Manifold C			
319	13	8.5	UP
320	12	0	UP
321	17	20.0	OP
322	12	21.0	OP
323	16	26.0	OP
324	9	5.5	UP
325	6	12.0	OP
326	3	11.5	OP
327	2	3.0	OP
328	4	0	UP
329	1	0.5	UP

Note: OP - Overpumped UP - Underpumped

conditions. This amount is the flow not intercepted by the underpumped wells under this scheme. Therefore, constant head nodes that exhibit on appreciable pumping rate indicate areas where additional wells would be a useful augmentation of the present system. Constant head nodes that exhibit a low discharge, indicate areas where additional wells would not be useful. Table 4.10 indicates areas where additional wells would not be useful in maintaining natural head differences across the slurry wall. In practice, these wells could be installed to aid in pumping more water out of areas that have been historically underpumped. By observing Table 4.10, it is apparent that the additional wells would be most useful in areas that have been historically underpumped, i.e., the eastern end of manifold A, the western end of manifold B and the western end of manifold C.

4.3 Transient Simulations Used in the Investigation of Overpumping to Attain a Reverse Gradient

The simulations performed thus far have been run under steady state conditions. Consequently, the potentiometric system changes very little during the model run. There is no change in storage, therefore the amount of water entering the model area must equal the amount of water leaving the model area. This has limited the versatility of the simulations performed up to now because schemes involving pumping over the equilibrium flow to the barrier (220 gpm) could not be evaluated. However, if storage is allowed to change, transient conditions in which pumping flucuates over time can be evaluated.

In the results obtained thus far, the model predicts that a reverse gradient cannot be attained for a total system flow of 220 gpm. However, it

Table 4.10
Locations and Flux Rates for
Additional Discharge Wells

Present Well	Additional Well	Node (Additonal Well)	Station along the Slurry Wall	Flux Rate for add'l well (GPM)
Manifold A				
330	1	199	192.0	0
	2	192	300.0	0
331	3	184	580.0	0
332	4	194	780.0	0
333	5	206	984.0	0
334	6	220	1200.0	0
335	7	232	1400.0	0
	8	235	1528.0	0
301	9	249	1680.0	0.5
	10	263	1808.0	0.5
302	11	279	1940.0	1.5
	12	288	2072.0	1.5
303	13	309	2184.0	5.0
	14	327	2304.0	3.0
304	15	344	2412,9	2.0
	16	363	2524.0	1.5
305	17	382	2632.0	2.5
	18	399	2732.0	1.5
306	19	414	2840.0	1.5
Manifold B				
307	20	429	2928.0	2.0
308	21	445	3096.0	3.0
309	22	463	3260.0	3.0
310	23	475	3432.0	5.5
311	24	486	3596.0	4.5
312	25	497	3756.0	3.5
313	26	509	3920.0	2.5
314	27	521	4084.0	1.0
315	28	537	4252.0	0.5
316	29	549	4420.0	0.5
317	30	562	4584.0	0.5
318	31	579	4756.0	1.0
Manifold C				
319	32	597	4920.0	2.5
320	33	615	5084.0	3.5
321	34	632	5260.0	3.0
322	35	651	5424.0	1.5
323	36	670	5592.0	2.0
324	37	693	5756.0	2.5
325	38	709	5992.0	1.0
326	39	736	6084.0	1.0
327	40	760	6248.0	1.0
328	41	783	6416.0	2.5
329	42	800	6584.0	0

is worth investigating how long the barrier system must be overpumped at its present configuration in order to attain a reverse gradient. The simulations performed under this scenario are discussed below. In both simulations, pumping and recharge is distributed to the wells according to natural interception proportions.

4.3.a. Overpumping 20 percent, Transient Simulation.

This simulation involves overpumping the barrier system 20 percent above the equilibrium barrier flow (220 gpm). Total pumpage under this scheme is 264 gpm. The total flow is proportioned to the manifolds according to the areas with the largest relative head differentials observed in previous simulations. Manifolds A and C have exhibited the highest positive head differentials in the simulations performed thus far. Therefore, these manifolds were overpumped the most. Flow was proportioned to each well based on natural interception. The individual rates are shown in Table 4.11. The results of this simulation indicate that after six months there is still a substantial positive gradient over manifold A. After a year of overpumping, a reverse gradient is attained over the entire barrier except for a few slightly positive areas in manifold A. In short, it will take an entire year of pumping at 20 percent over natural to attain a reverse gradient.

Table 4.11
Discharge Well Rates Used in Overpumping Simulations

Well	4.3.a overpump 20%	4.3.b overpump 40%
Manifold A		
330	2.5	3.0
331	2.0	2.5
332	1.0	1.5
333	0	0
334	0.5	0.5
335	1.5	1.5
301	3.0	3.5
302	5.5	6.0
303	11.0	13.0
304	19.0	21.0
305	20.0	22.0
306	14.0	15.5
	80 gpm	90 gpm
Manifold B		
307	5.5	7.0
308	6.0	8.0
309	6.0	7.5
310	7.5	9.5
311	5.0	6.5
312	5.0	6.0
313	8.0	10.5
314	8.0	10.5
315	3.5	4.5
316	3.5	5.0
317	5.0	6.0
318	7.0	9.0
	70 gpm	90 gpm
Manifold C		
319	15.5	17.5
320	14.5	16.5
321	20.0	22.5
322	14.0	16.0
323	20.0	22.0
324	11.0	12.0
325	7.5	9.0
326	4.0	4.5
327	2.0	2.0
328	5.0	5.5
329	0.5	0.5
	114 gpm	128 gpm
TOTAL	264 gpm	288 gpm

4.3.b. Overpumping 40 percent, Transient Simulation

This simulation involves overpumping the barrier system 40 percent above equilibrium (288 gpm total). The individual rates are shown in Table 4.11. The results indicate that at six months, a positive gradient still exists over manifold A; although not as substantial as that estimated in 4.3.a. After a year, a complete gradient reversal is attained and is more substantial than in 4.3.a.

It should be noted that these simulations do not account for the actual irregularities in discharge well pumping rates occurring during operation. In the simulations, it is assumed that the total barrier flow and individual well rates is maintained at a constant value throughout the period of operation. In actuality this does not occur. Consequently, the model predictions might be overestimations.

4.4 Utilization of Existing System Augmented by Recharge Trenches

Although the reconfiguration schemes discussed above result in large reductions in head differential, none of the simulations have resulted in a complete and substantial gradient reversal over the entire length of the barrier. As stated before, the natural gradient in this region of the Arsenal appears to limit the ability to attain a reversal.

Consequently, it was decided to investigate the potential for attaining a gradient reversal over the section of the barrier that intercepts the most highly contaminated groundwater. This investigation was prompted by the Arsenal's desire to augment the recharge system west of Building 808. The

Arsenal has proposed the use of recharge trenches over the entire length of manifold A which includes the original pilot scale system. The trenches have been proposed as an alternative to recharge wells in the area in the hope of attaining a reverse gradient in this region and avoiding the problem of recharge well clogging. The proposed scheme involves the installation of a line of ten trenches installed 45 feet from the slurry wall along Manifold A and the western part of Manifold B. Each trench would be 160 feet long - 3 feet wide and excavated to a depth of 20 feet. (Morris-Knudson, 1987).

Various recharge schemes involving different amounts of trench recharge were modeled to determine the effect on the head differential across the slurry wall. The trenches were modeled as fully-penetrating point recharge sources distributed across manifold A. Table 4.12 lists the nodes that correspond to each trench in the model. The individual recharge trench

Table 4.12

Nodes Used for Trench Modeling

	Trench	Node No.	Station along the Slurry Wall (Feet)
West	1	943	192
	2	952	580
	3	968	984
	4	1000	1400
	5	1039	1680
	6	1092	1940
	7	1200	2304
	8	1281	2632
	9	1336	2928
East	10	1368	3260

simulations are discussed below. In each simulation, the discharge system was operated at natural equilibrium manifold rates. Water was distributed to the individual wells according to historical proportioning. The assumption is that the wells would continue to pump at historical rates while the manifolds tend toward natural equilibrium.

4.4.a. Maximum Trench Flow Simulation

This simulation was performed to determine the theoretical maximum amount of recharge that would flow through the ten trenches assuming the recharge wells in the same region are shut off. The amount was determined by setting the head at the nodes corresponding to the trenches to the maximum head that could be accommodated by the trench. The maximum head was determined to be four feet below the ground surface. As indicated in the trench specifications (Morrison-Knudson, 1987), at this depth, a lateral recharge pipe would be installed in each trench and used for conveying water to the trenches. By setting the head to a constant value at this level, the model calculated the amount of water that would be required to maintain the specified head in the trench. Table 4.13 lists the trench recharge rates that resulted from this simulation. The results indicate that a maximum of 166 gpm may be recharged under maximum head conditions. It should be noted that this computation does not account for the potential clogging of the pores within the trench by carbon fines. Consequently this scenario is idealized. Table 4.13 indicates that very little water will be recharged in trenches one through five. These trenches are located in the far west region of the barrier where the aquifer is thin and an estimated three percent of total groundwater flowing to the north boundary is accounted for in this region. The recharge potential of the trenches in the region are

Table 4.13

Maximum Trench Flow Rates

Trench #	GS elevation	WT head	Rate (GPM)
1	5154.5	5150.5	1.83
2	5151.5	5147.5	3.26
3	5148.0	5144.0	1.48
4	5146.0	5142.0	3.19
5	5144.0	5140.0	0.00
6	5147.0	5143.0	10.87
7	5146.0	5142.0	30.82
8	5146.0	5142.0	0.00
9	5152.3	5148.3	97.44
10	5148.0	5144.0	17.73

Total Trench Flow = 166 GPM (Maximum)

limited to the natural pre-barrier groundwater flow capacity of this region. The bulk of recharge is conveyed through trenches six through ten. These trenches are located in and around one of the paleochannels that intersect the barrier. Trenches five and eight show a zero recharge capability when trenches are set to maximum head. The reason for the zero values is due to the difference in ground surface and therefore maximum head values between adjacent trenches. Between trenches four and six the ground surface appears to dip to the level of trench five. Consequently, there is a gradient toward trench five. Similarly trench nine has a much higher head then trench eight causing a gradient toward trench eight. In practice, the heads in adjacent trenches would be maintained at similar values to allow a more equitable distribution of recharge and to avoid substantial gradients

between trenches. This simulation represents the theoretical maximum recharge capacity of the ten recharge trenches.

To determine the effect of the trenches recharging at maximum capacity on the head differential across the slurry wall, a simulation was performed in which the flux rates shown in table 4.13 were specified at the corresponding trench nodes. In order, to allow 166 gpm of recharge through the trenches, the amount of water recharging through the wells in manifold B and C was reduced so that total recharge is 220 gpm. The discharge wells were pumped at the historical proportioning while the manifolds were pumped at natural equilibrium assuming that the manifolds would tend toward natural conditions. The rates are listed in table 4.13. Figure 4.17 illustrates the head differential along the slurry wall resulting from this simulation. As can be observed seen, the head differential is substantially negative over all of manifold A and half of manifold. Conversely, the head differential is substantially positive over all of manifold C and half of manifold B. The average head differentials for manifolds A, B, and C are -5.56, 1.21 and 8.81 respectively. Although a substantial positive gradient still exists over half of the barrier, this scheme accomplishes the objective of reversing the gradient over manifold A.

4.4.b. Trenches recharging flowing at 108 gpm

This simulation represents a more practical scheme in that a more equitable distribution of heads among the trenches results. Table 4.14 lists the water table heads and the flux rates for this trench simulation. As can be seen, there are no major gradients between trenches; therefore,

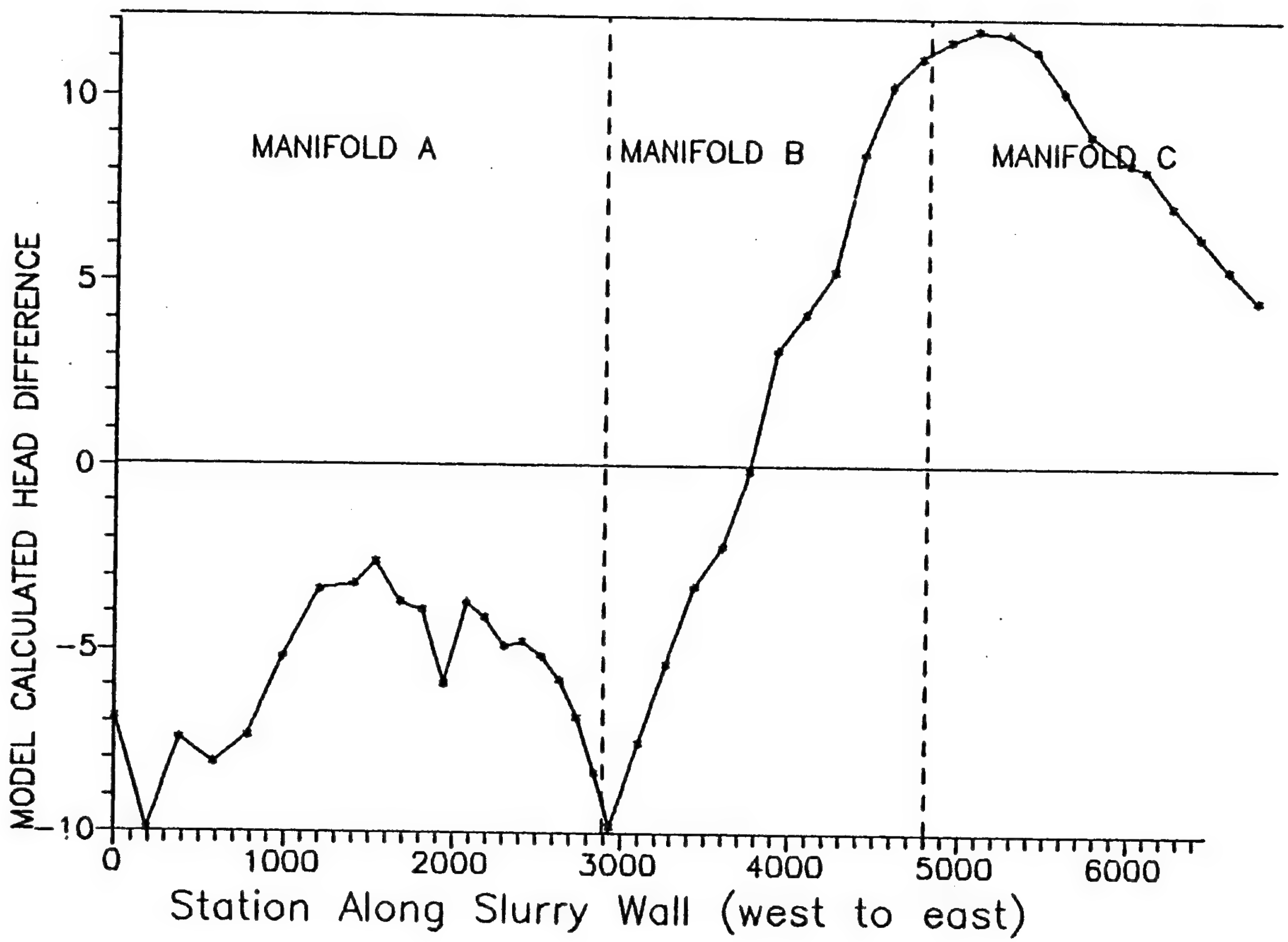


Figure 4.17 Estimated Head Differential Distribution, Trench Flow=Maximum

Table 4.14

Trench Flow Rates

Total Trench Flow = 108 GPM (Maximum)

Trench #	GS elevation	WT head	Rate (GPM)
1	5154.5	5145.4	1.00
2	5151.5	5142.5	1.80
3	5148.0	5139.9	1.00
4	5146.0	5138.2	2.70
5	5144.0	5139.8	1.80
6	5147.0	5141.0	8.10
7	5146.0	5138.8	22.60
8	5146.0	5139.6	13.60
9	5152.3	5141.7	29.40
10	5148.0	5141.8	26.40

Table 4.15

Trench Flow Rates

Total Trench Flow = 75 GPM (Maximum)

Trench #	GS elevation	WT head	Rate (GPM)
1	5154.5	5143.9	0.50
2	5151.5	5143.3	1.00
3	5148.0	5139.4	1.00
4	5146.0	5137.9	2.70
5	5144.0	5135.8	0.50
6	5147.0	5137.1	4.50
7	5146.0	5136.4	16.30
8	5146.0	5137.3	11.30
9	5152.3	5139.1	19.00
10	5148.0	5139.4	18.10

Table 4.16

Trench Flow Rates

Total Trench Flow = 38 GPM (Maximum)

Trench #	GS elevation	WT head	Rate (GPM)
1	5154.5	5143.8	0.50
2	5151.5	5137.7	1.00
3	5148.0	5135.4	0.50
4	5146.0	5133.5	1.00
5	5144.0	5134.9	0.00
6	5147.0	5136.8	4.50
7	5146.0	5133.8	9.00
8	5146.0	5134.5	4.50
9	5152.3	5136.0	10.00
10	5148.0	5136.6	7.20

each trench contributes to the total recharge. Figure 4.18 illustrates the head differential for this simulation. A reverse gradient still exists over manifold A. As with simulation 4.4.b recharge was reduced for wells in manifolds B and C to result in a total recharge of 220 gpm. Consequently a substantial positive gradient still exists over manifolds B and C. The average head differentials for manifolds A, B and C are -2.07, 1.41 and 5.85 feet respectively.

4.4.c Trenches Recharging at 75 GPM

Table 4.15 lists the water table heads and trench flux rates utilized in this simulation. Figure 4.19 indicates that when trench recharge is reduced to 75 gpm, a positive head differential results over manifold A. Consequently recharging the trenches at 75 gpm or below would not achieve the objective of the trench scheme. The average head

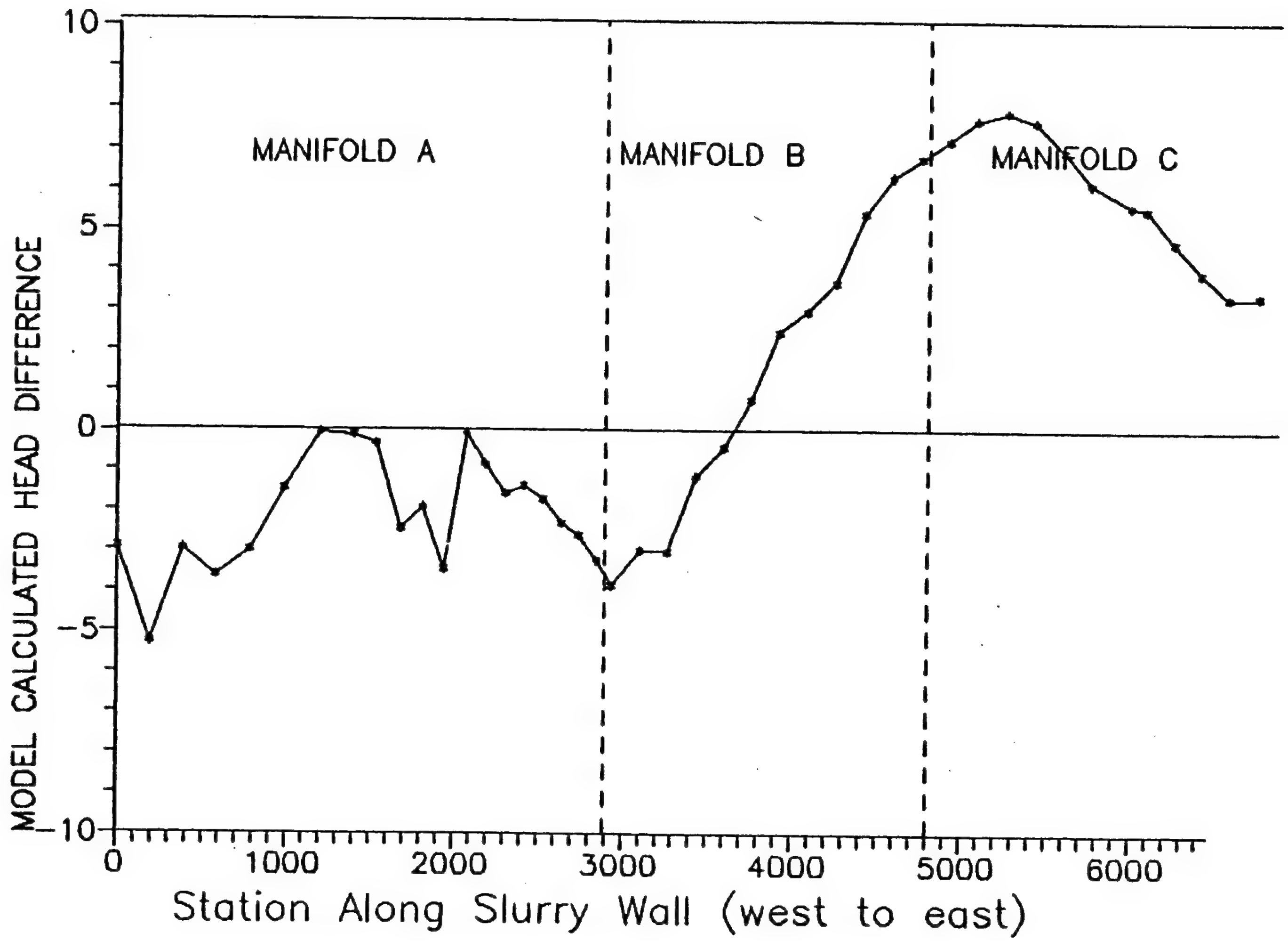


Figure 4.18 Estimated Head Differential Distribution,
Trench Flow = 108 GPM

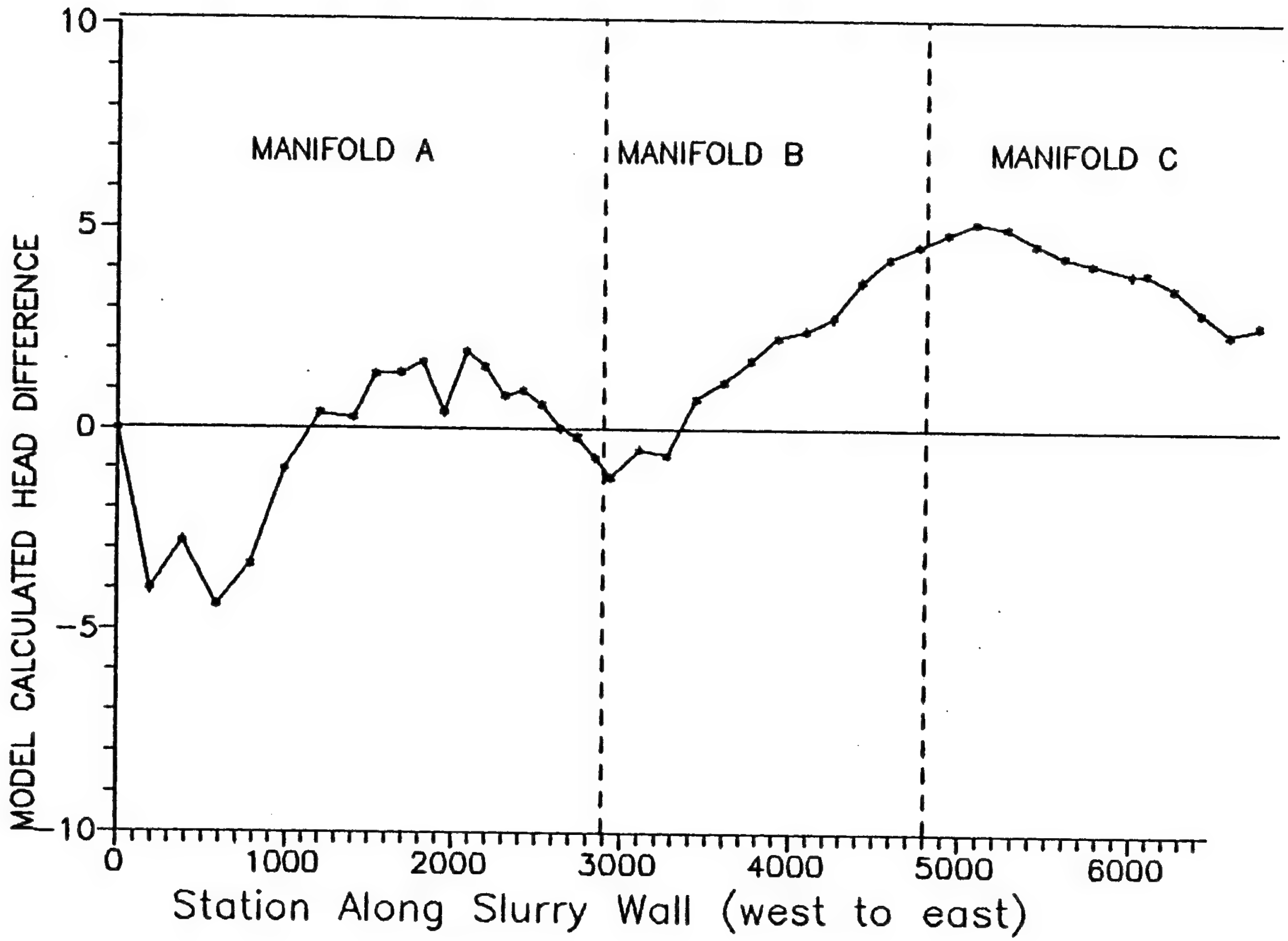


Figure 4.19 Estimated Head Differential Distribution,
Trench Flow = 75 GPM

differential for manifolds A, B and C are -0.24, 1.79 and 3.98 respectively.

4.4.d Trenches Recharging at 38 GPM

Table 4.16 lists trench recharge rates used in this simulation. Figure 4.20 illustrates the resulting head differential along the slurry wall. The results indicate that as trench flow rates approach the below natural equilibrium rates (currently exhibited at the Arsenal), the gradients across the slurry wall increase to large positive values. The average head differentials for manifolds A, B and C are 2.75, 2.49 and 1.52 feet respectively.

4.4.e Utilization of Both Trenches and Recharge Wells Along Manifold A

This simulation was performed to determine the effect of utilizing the present recharge wells and the trenches simultaneously. The wells were recharged at historical proportioning while the trenches were recharged at the rates shown in Table 4.17. The resulting head differential is illustrated in Figure 4.21. The results indicate that a negative head differential may be attained over manifold A by implementing this scheme. The average head differentials for Manifolds A, B and C are -3.8, 2.1 and 6.0 feet respectively. Although this scheme may seem to be an over-design, it does have a practical application. Operations data indicate that there is difficulty in recharging sufficient amounts in manifold A and that excess water that cannot be recharged here is discharged to the bog located in the east end of manifold B. Instead of discharging this water to the bog, the water may be pumped to the trenches, thus helping to reverse the gradient over the region of the barrier that intercepts the contaminant plumes.

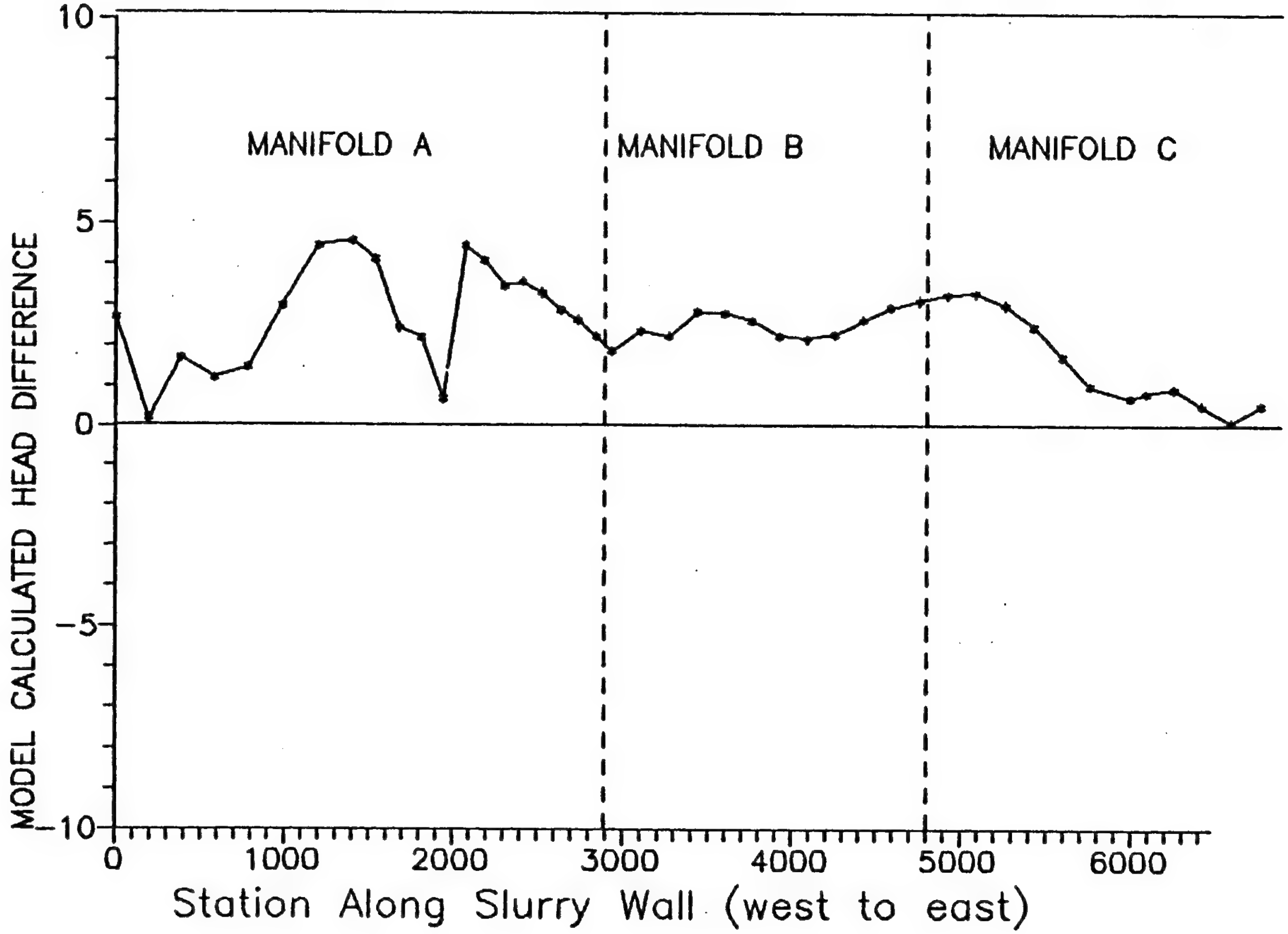


Figure 4.20 Estimated Head Differential Distribution, Trench Flow = 38 GPM

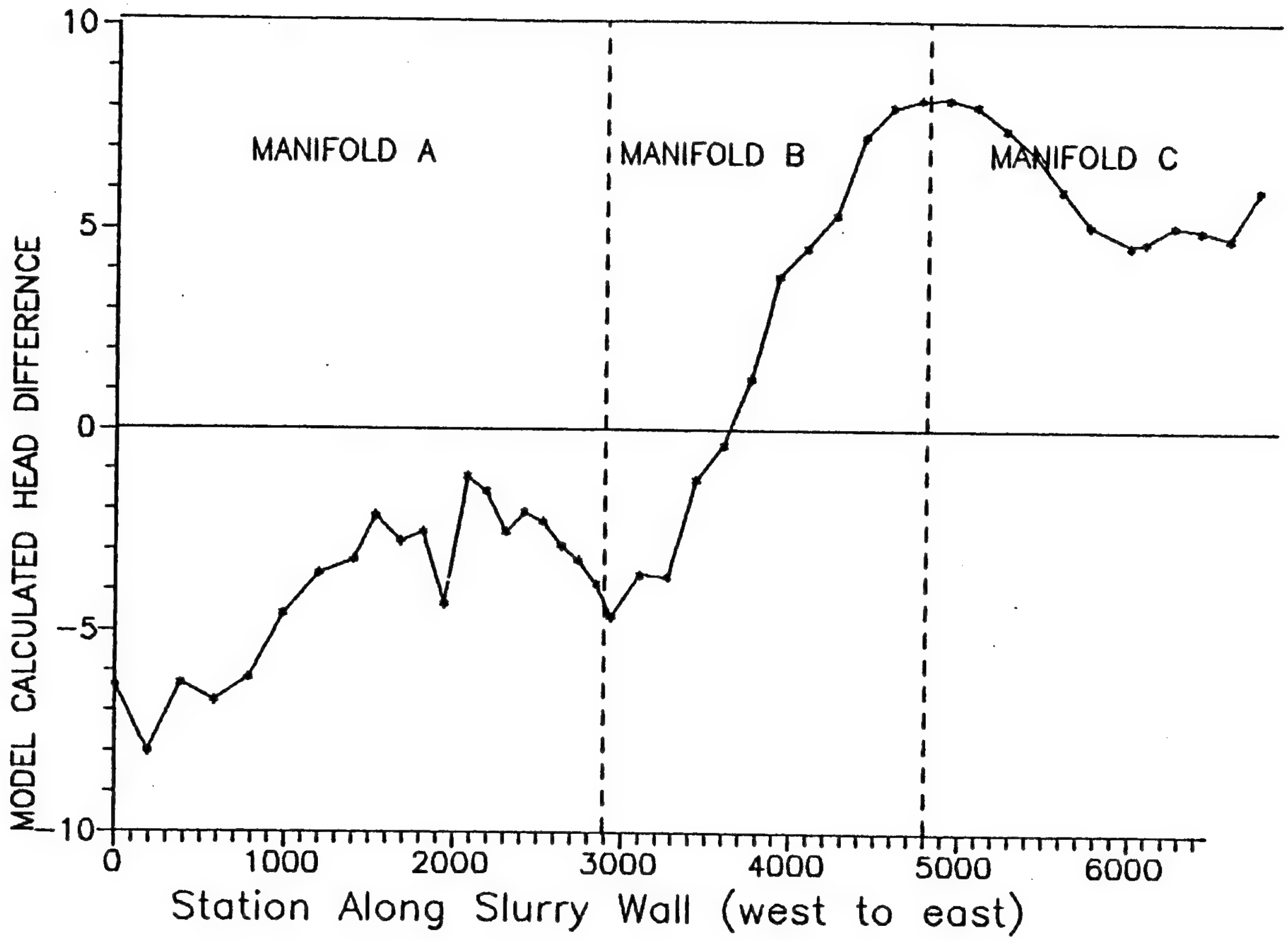


Figure 4.21 Estimated Head Differential Distribution, Utilization of Both Trenches and Wells

Table 4.17

Trench Flow Rates

Total Trench Flow = 113 GPM

Trench #	GS elevation	WT head	Rate (GPM)
1	5154.5	5148.6	1.00
2	5151.5	5146.2	1.80
3	5148.0	5143.4	1.00
4	5146.0	5141.6	2.30
5	5144.0	5141.0	1.00
6	5147.0	5142.9	10.10
7	5146.0	5140.9	23.20
8	5146.0	5141.3	12.60
9	5152.3	5143.4	31.40
10	5148.0	5142.9	28.60

NOTE: Trenches used in addition to the present well system in this simulation.

Table 4.18 lists the average head differences for all the trench simulations discussed herein. The averages for a simulation in which the present configuration is being operated at manifold C overpumping fluxes distributed to the wells according to historical proportions (table 4.6) was included for comparison. A comparison of these averages indicates that in all schemes involving the trenches flow at some appreciable rate, the gradient across manifold B is a greatly improved over that resulting from use of the present system. The model estimates that if the proposed trench scheme is recharged at over 100 gpm, a substanial reverse gradient can be expected over the entire length of manifold A.

It should be noted here that the model estimates the water table under ideal recharge conditions. That is, the occurrence of clogging by carbon fines is not accounted for. Clogging has been observed in the current

recharge wells. Clogging could also, effect the recharge capability of the recharge trenches. Furthermore, localized cementing in the alluvium could also hamper recharge efforts. These possibilities should be considered in determining the feasibility of any trench scheme.

Table 4.18 Comparison of Estimated
Average Head Differential, Trench Simulations

Scenario	Manifold A	Manifold B	Manifold C
Trench Flow (GPM) 166	-5.56	1.21	8.81
108	-2.07	1.41	5.85
75	-0.24	1.79	3.98
38	2.75	2.49	1.52
Trenches and wells (Trench flow rate = 114 gpm)	-3.80	2.10	6.01
Wells at Historical Rates (Flouride Dilution Scenario)	6.0	3.0	-0.25

4.5 Investigation of Operational Breakdown Scenarios

The Arsenal has posed questions concerning the hydraulic gradient across the barrier system during treatment plant shutdown. Plant shutdown has occurred in the past at the Arsenal due to power outages and maintenance problems. Most plant shutdowns have been reported to last no more than two to three days in order to service broken parts or clean filters (Ward 1988). These occurrences have created a need by the Arsenal to predict the barrier system performance when subject to plant shutdown. The predominant questions here are when and where might the groundwater overtop the barrier during prolonged plant shut down.

When the treatment plant shuts down, it can no longer accept the contaminated groundwater pumped from the discharge wells. Therefore, the discharge wells are shut off causing subsequent build up of water against the slurry wall. This model investigation involved determining the amount of time after the wells are shut off that the water levels reach the ground surface. This time period was estimated by running transient simulations and comparing water levels to ground surface elevations after each time step. Ground surface information was obtained from Arsenal topographic maps and an as-built elevation profile of the top of the slurry wall. Ground surface elevations were determined for each nodes within 1000-1500 feet of the upgradient side of the barrier. Three nodal points comprise each element. The three corresponding ground surface elevations were averaged over each element and compared with head values averaged in the same way. This comparison occurs after each time step. This comparison scheme was employed in each breakdown simulation in determining overtopping times and areas. In each simulation discharge wells were pumped for an initial six

month period and instantaneously shut off to simulate plant or well failure. Each simulation is discussed below.

4.5.a. Failure of the Treatment Plant While Flowing at 220 GPM

This simulation was performed to estimate the effect on the barrier system of treatment plant failure while being operated 220 gpm. This particular flow rate was chosen because it represents the May 1987 natural flow to the barrier. The results of this simulation indicate that groundwater reaches the ground surface after 12 days of plant failure and that overtopping occurs adjacent to the barrier in the area near First Creek. Historically, the plant has never been down for 12 days. However, results of extended shutdown scenarios are desired by the Arsenal to help plan breakdown strategies.

4.5.b. Failure of Manifold C Wells While the Plant is Flowing at 220 GPM

This simulation was performed to determine the effect of the failure of pumping wells in the region of the barrier most vunerable to overtopping. As stated before, the most vunerable region has historically been near First Creek and the First Creek paleochannel located in manifold C. This breakdown scenario involves the failure of manifold C wells only. Manifold A and B wells remain pumping at the present manifold C overpumping scenario with pumpage distributed to the wells based on calibrated proportions. The rates are listed in Table 4.19. The results indicate that overtopping occurs near First Creek after 16 days of shut down of Manifold C. The longer duration here versus the duration estimated in simulation 4.4.a. appears to be caused

Table 4.19

Discharge well Flux Rates Used in each Breakdown Simulation

Discharge Well	Discharge Well Flux Rates (GPM)		
Manifold A	4.4.a	4.4.b	4.4.c
330	0	2.0	2.0
331	0	1.0	1.0
332	0	2.0	2.0
333	0	0.50	0.5
334	0	0.50	0.5
335	0	0.50	0.5
301	0	1.5	1.5
302	0	2.5	2.5
303	0	0	0.0
304	0	16.5	16.5
305	0	11.0	11.0
306	0	17.0	17.0
	0	55.0	55.0
Manifold B			
307	0	0	0
308	0	0	0
309	0	0	0
310	0	0	0
311	0	0	0
312	0	0	0
313	0	16.0	22.0
314	0	17.0	24.5
315	0	7.0	10.0
316	0	10.0	14.5
317	0	7.0	10.0
318	0	13.0	19.0
	0	70.0	100.0
Manifold C			
319	0	0	0
320	0	0	0
321	0	0	0
322	0	0	0
323	0	0	0
324	0	0	0
325	0	0	0
326	0	0	0
327	0	0	0
328	0	0	0
329	0	0	0
TOTAL	0	0	0

by pumpage by manifold B. Pumping manifold B during manifold C shutdown appears to relieve the head build up in Manifold C somewhat, causing overtopping to occur over a slightly longer period of time.

4.5.c. Failure of Manifold C wells, overpumping Manifold B wells.

The simulation is the same as simulation 4.5.b. except that manifold B wells are overpumped in an attempt to compensate for Manifold C line failure. Manifold B wells are pumped at approximately 40 percent over natural interception. The individual well rates are listed in Table 4.19. The results indicate that water overtops the slurry wall at approximately 18 days after failure of manifold C wells in the area near First Creek. It is apparent from the comparison of these results with those of 4.5.b. that overpumping manifold B provides increased relief during manifold C failure.

4.5.d. Failure of Discharge Wells 319, 320, 321, 322 While the Plant is Being Operated at 220 GPM

Discharge wells 319, 320, 321, and 322 have historically been productive wells. This simulation was performed to estimate the effect of failure of these wells at a normal stable plant flow of 220 gpm. The model estimates that overtopping will occur near First Creek after 89 days of plant shutdown. This is a rather long period of time and it is likely that these wells will never shut down for this amount of time without some type of corrective action taken. However, this simulation indicates that the productivity of these wells are important to the success of the barrier system in this area.

4.5.e. Failure of the Treatment Plant while operated at 300 GPM

This simulation was performed to estimate the effect of treatment plant failure, if previous to failure, there was a management decision to operate the system at 300 gpm. The individual well rates are listed in table 4.20. The barrier was operated at 300 gpm for an extended period of time before failure. Under this scenario, the model estimates that failure will occur near First Creek after 34 days of plant shutdown. This time period is approximately three times longer than that estimated in simulation 4.5.a. The reason for the increased duration is that greater drawdown near the slurry wall exists at the time of plant failure. Consequently, it took longer for the heads to build up and rise to the ground surface.

4.5.f. Failure of Manifold C Wells while the plant is being operated at 300 gpm

As in simulation 4.5.b., this simulation was performed to determine the effect of Manifold C failure. However here, the potentiometric surface at the time of failure is lower due to the increased pumpage. The model estimates that overtopping will occur in the area near First Creek after 62 days of plant shut down. This duration is almost two times the duration in the previous simulation. The reason for the difference is that manifold A and B wells remain pumping at a substantial rate (160 GPM total). The individual well rates are listed in Table 4.20. The total flow after failure is high enough to compensate for the Manifold C failure and to prevent overtopping for a substantial period of time.

Table 4.20

Discharge Flux Rates Used in 300 GPM

Breakdown Scenario at Time of Failure

Discharge Well	Flux Rates		
Manifold A	4.4.e	4.4.f	4.4.g
330	0	2.0	2.0
331	0	1.0	1.0
332	0	2.0	2.0
333	0	0.50	0.5
334	0	0.50	0.5
335	0	0.50	0.5
301	0	2.0	2.0
302	0	2.5	2.5
303	0	0	0.0
304	0	18.0	18.0
305	0	12.0	12.0
306	0	19.5	19.5
	0	60.5	60.5
Manifold B			
307	0	0	0
308	0	0	0
309	0	0	0
310	0	0	0
311	0	0	0
312	0	0	0
313	0	22.5	22.5
314	0	24.5	24.5
315	0	10.0	10.0
316	0	14.5	14.5
317	0	10.0	10.0
318	0	19.0	19.0
	0	100.5	100.5
Manifold C			
319	0	0	0
320	0	0	0
321	0	0	0
322	0	0	0
323	0	0	33.5
324	0	0	7.5
325	0	0	16.0
326	0	0	14.5
327	0	0	4.0
328	0	0	0.0
329	0	0	0.5
TOTOL	0	0	76.0

4.5.g. Failure of Discharge Wells 319, 320, 321, 322 while the plant is being operated at 300 GPM.

Similar to simulation 4.5.d, this simulation was performed to estimate the effect of failure of wells 319, 320, 321, and 322 at a plant flow of 300 gpm. The model estimates that overtopping will likely never occur under this scenario. The reason is that at failure, the surrounding wells remain pumping at increased rates. The end result is that total plant flow is only decreased to 237 gpm. This is still above the natural flow to the barrier and is more then enough to compensate for the failure of wells 319, 320, 321, and 322.

4.5.h Plant Shutdown at Operation Rate of 220 GPM, Flow to the Barrier increased

This simulation was performed to estimate the effect of treatment plant shutdown during a period of higher-than-normal flow to the barrrier. This flow scenario might be expected during late winter and early spring months. The simulation involved increasing the boundary flux at the south end of the model area to the point where flow to the barrier increased to approximately 300 gpm. Other conditions remained the same as in previous simulations. Under these conditions, the model estimates that overtopping will occur near First Creek after eight days of plant shutdown. A comparison of this result with that of simulation 4.5.a, indicates that overtopping occurs four days sooner under high water table conditions. Thus, overtopping times may vary with season of the year.

4.5.i. Plant Shutdown at Operation Rate of 300 GPM, Flow to the Barrier Increased.

To compare with the previous simulation and simulation 4.5.e., this simulation was performed to estimate the effect of overpumping and subsequent plant shutdown during high water table conditions. Here, the plant is operated at 300 gpm. The model estimates that overtopping will occur near First Creek after 25 days of plant shutdown. This duration is 9 days less than pumping at 300 gpm under normal water table conditions (simulation 4.5.e.). Again, this indicates that overtopping times may vary with season of the year. The results of all Breakdown simulations are presented in Table 4.21.

Table 4.21
Overtopping Times and Locations

Simulation	Overtopping Time (Days)	Overtopping location (Station along the Slurry Wall, Feet)
4.4.a Total Failure at 220 gpm	12	5400-6000
4.4.b C line Failure at 220 gpm	16	5400-6000
4.4.c C line Failure at 220 gpm, overpumping Manifold B	18	5400-6000
4.4.d Failure of 319, 320, 321, 322 at a Plant Flow of 220 gpm	89	5400-5800
4.4.e Total Failure at 300 gpm	34	5400-6000
4.4.f C line Failure at 300 gpm	62	5400-6000
4.4.g Failure of Wells 319, 320, 321, 322 at 300 gpm	Overtopping does not occur	

CHAPTER 5

THREE DIMENSIONAL CONTAMINANT TRANSPORT MODELING

This phase of the study (referred to as the 3-D study involved a three dimensional modeling investigation of pathways of contaminant migration in the vicinity of the Barrier. The focus of this phase is on vertical flow patterns and mechanisms for contaminant migration into the Denver sand units. The investigation involved an evaluation of local vertical groundwater gradients as well as the effects produced by historic pumping of the Denver Formation Wells located near the North Boundary Barrier System to provide insight as to why underlying sand lenses are contaminated. Unlike the two-dimensional study, the data available for this study was very limited. Data such as potentiometric head, hydrogeologic characteristics of the Denver formation as well as Denver well pumping rates were minimal. Periodic data (month/yearly) needed for model calibration and verification does not exist.

The most plentiful and quality information on the Denver formation was provided by Environmental Science and Engineering (ESE, 1988). However, their data characterized only a small part of the Denver formation in the immediate vicinity of the North Boundary Barrier System.

Due to the limited amount of data available, the objectives of this investigation were limited to the following:

1) Theorize a potential contaminant migration pathways into the Denver formation and;

2) Establish the feasibility of a more indepth and detailed 3-D modeling study.

To fulfill these objectives, the study focused on the interaction between the uppermost Denver Sand unit (indicated to be NBM#1A in ESE 1988) and the alluvial aquifer.

The discussion in this chapter includes the 3-D model used, model input data, and model simulations and results.

5.1 The Computer Model

The model used in the 3-D study is a quasi three dimensional finite element model. It is a modified version of the two-dimensional flow and transport code CSU-GWTRAN written by Dr. James Warner. The modifications were performed and tested by (Bahadur, 1988). The model CSU-GWTRAN was modified to allow digital simulation of multi-layer aquifer systems. The model is referred to as a quasi 3-D model because the assumption is that one-dimensional vertical leakage through the aquifer links the upper and lower aquifers. Flow in the aquifer is assumed to be completely horizontal (two dimensional) whereas flow in the aquitard is assumed to be completely vertical (one dimensional). In contrast, fully 3-D models utilize 3-dimensional solution techniques which allow flow in any layer to occur in any direction.

Linear finite elements are used to link the two-dimensional triangular elements of the upper and lower layers in the model. Linking occurs at each node, thus the same finite element mesh is used for each layer in the model. The model calculates leakage through the confining layer by using the heads in the upper and lower aquifers as boundary conditions and utilizing the

finite element formulation to solve flow equation for heads in the aquitard. A similar scheme is used to solve the one dimensional dispersion equation for solute transport through the aquitard. Flow and solute transport in the upper and lower layers in the model are solved for separately. Vertical flow and transport through the confining layer are incorporated as a source/sink term in the 2-D equations.

The quasi 3-D model has been tested by Bahadur (1988) for a number of multilayer aquifer cases and has compared well with a number of analytical solutions. For a detailed description of the model and the test cases performed refer to Bahadur (1988).

5.2 Model Data and Assumptions

As stated before, the amount of data characterizing the Denver Formation are limited; due in part, to the fact that historically the alluvial aquifer has been studied the most because it is more likely to contain contaminants. Environmental Science and Engineering (ESE) has evaluated flow and contaminant transport in the Denver Formation and has compiled the most current data set. ESE (1988) was the primary source of data for the 3-D study. Numerous other sources of information contain data on the Denver Formation, most of which are discussed in ESE (1988).

5.2.a The Finite Element Mesh

The 3-D finite element mesh is depicted in Figure 5.1. It represents a sub-area of the two dimensional model.

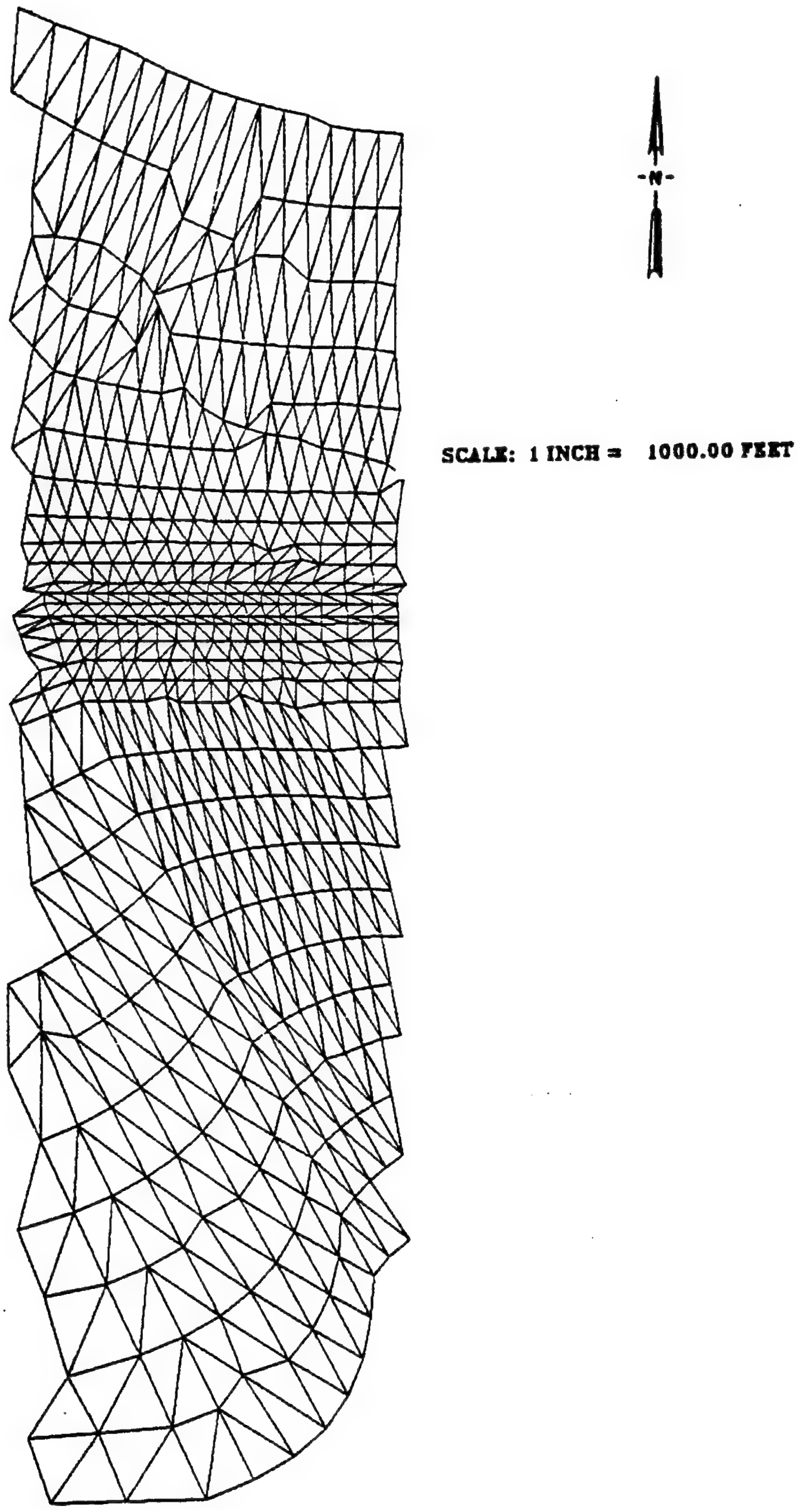


Figure 5.1
The 3-D Finite Element Mesh

The 3-D mesh was generated from the 2-D mesh. The nodes and elements in the 3-D model are located in the precise location as in the 2-D model; however, they posses different nodal and element numbers. In this manner the calibrated 2-D model was used as the upper layer in the 3-D model. Thus, only the bottom layer (Denver Formation) and the confining layer required characterization.

The limits of the mesh used in the 3-D model included in the part of the North system barrier that comprised the original pilot scale system. This area has been found to be contaminated up and down gradient of the slurry wall.

The mesh shown in Figure 5.1 includes 518 nodes and 942 elements all of which were part of the mesh used in the 2-D study. The total area represented by the mesh is 0.72 square miles, which is approximately 26 percent of the 2-D model area. Nodal spacing varies from 45 feet in the vicinity of the barrier system to 850 feet in the off-post area to the north. As with the 2-D study the density of elements is greatest in the vicinity of the barrier which allows high resolution in this area.

The part of the North barrier system modeled in this study includes approximately 2720 feet of the slurry wall. The alluvial dewatering wells included in the model area are dewatering wells 334 and 335 and 301 and 310. The recharge wells include wells 437 and 438 and 401 through 416. There were eleven Denver dewatering wells modeled in this study which include wells 336 through 346. The part of the barrier system included in the 3-D model area is shown in Figure 5.2.

Each alluvial well included in the 3-D study is located at the same node as it was in the 2-D mesh. The Denver dewatering wells are represented by the line of nodes between the nodes representing alluvial wells and the

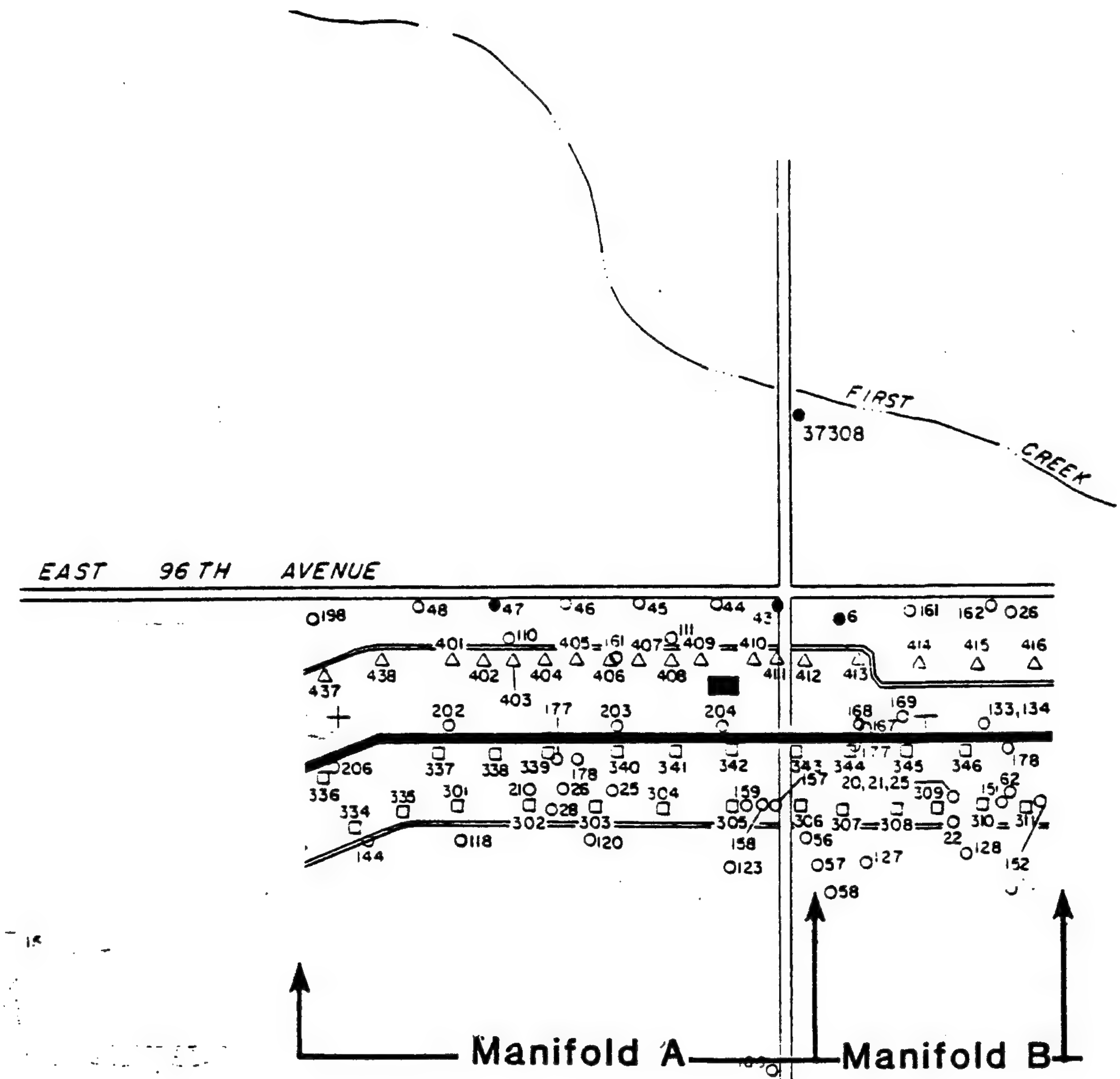
barrier nodes. It must be kept in mind that while the finite element mesh is the same for the top and bottom layers, the nodes and elements of each mesh represent the characteristics of their respective layers.

5.2.b Boundary Conditions

For the top layer of the 3-D model (alluvial aquifer) the north and south boundaries were modeled in the same fashion as in the 2-D study. The boundary conditions for the top layer were chosen so that the shape of the potentiometric surface computed in the 2-D study was preserved. The south boundary represents a contact between the alluvium and the Denver formation. Consequently, this area was modeled as constant head or in some simulations as a specified flux boundary. The east and west boundaries were modeled as no-flow boundaries since the predominant flow direction is parallel to these boundaries. The bottom layer in the model (Denver Sand Unit NBW#1A) used the same general boundary conditions as the top layer (alluvial aquifer). Very little data were available on the Denver Formation.

5.2.c Potentiometric Surface

The Potentiometric Surface used for the alluvial aquifer (top layer) is the May 1987 model calculated surface. The potentiometric surface used for the Denver Sand unit (lower layer) was interpreted from the a map constructed by ESE for Spring 1987, ESE (1988). However, their investigation focused on a limited part of this sand unit (approximately 500-1000 feet both up and downgradient of the slurry wall). There was no data available for the remaining part of the model area. ESE (1988) reported that in the are studied the potentiometric surface in NBW#1A is



ALLUVIAL WELLS	DENVER WELLS	ALLUVIAL WELLS
301,302,303,304	336,337,338,339	307,308,309,310
305,306,330,331	340,341,342,343	311
332,333,334,335	344,345,346	

Figure 5.2

Portion of Barrier System
Modeled in the 3-D study

very similar to the potentiometric surface of the alluvial aquifer. With this observation in mind, the initial potentiometric head of the Denver sand unit inputed into the model values as the top layer. Although this assumption cannot be verified, the approximation is adequate in light of the study objectives.

5.2.d Aquifer Thickness

For the alluvial aquifer (top layer),the aquifer thickness values used were the May 1987 2-D model computed values. For the Denver sand unit, (NBW #1A) the average aquifer thickness is 13.57 feet This average aquifer thickness was used as a uniform value in the model for the lower layer. A uniform value was used in the model because a detailed characterization of the Denver units does not exist.

The clay shale layer separating the alluvium and the top Denver Sand unit is of an average of 8 feet thick (Earth Sciences Associates 1980). In the vicinity of the pilot scale system, the layer is reported to be fractured. A uniform value of 8.0 feet was used as the average thickness of the confining layer.

5.2.e Storage coefficient/Specific Yield

For the alluvial aquifer the (top layer) the storativity values used were the same as the corresponding valuesin the 2-D model. The storage coefficient for the Denver Sands unit was determined from several pump tests performed by ESE (1988). These pump tests had a range in storage coefficient values for the Denver Sands unit from 0.000026 to 0.0051. A storativity of 0.004 was used in the model for the Denver sand unit (lower layer).

5.2.f Hydraulic Conductivity and Transmissivity

For the alluvial aquifer (top layer), the transmissivity used was obtained from the calibrated 2-D model. For the Denver Sand unit (lower layer), an average value obtained from pump tests (ESE, 1988) of 4.3 ft^2/day (32.17 gal/day/ft) was used.

For the weathered clay shale (confining layer) a vertical hydraulic conductivity value of 3.07 gpd/ft^2 (4.104×10^{-5} ft/day) (ESE 1988) was used. This value was determined from a test performed in the weathered clay shale by Black and Veatch.

5.2.g Porosity

Porosity is used in the calculation of pore velocities for contaminant transport. Previous modeling studies by Konikow, Robson and Warner have used a porosity of 0.30 for the alluvium. This value was used in this study.

The porosity for the Denver Sand unit was obtained from a void ratio analysis (ESE 1988). From this analysis, a porosity value of 0.33 for medium to coarse-grained sandstones was obtained. This value was used for the lower layer in the model.

The porosity value for the confining clay shale layer used in the model was 0.43 (ESE 1988).

5.2.h Alluvial and Denver Well Rates

The pumping rates for the alluvial wells were obtained from the 2-D model and are given in Table 4.6. About 36.15 gpm were pumped from the alluvial wells located in the 3-D model study area.

Part of this 3-D modeling effort involved the investigation of contaminant transport induced by pumping of wells perforated in the upper part of the Denver Formation. Information regarding Denver well pumping rates was nonexistant. The only information on Denver well rates was obtained from personal communication with Arsenal personnel who observed the operation. Arsenal personnel have indicated that the Denver well pumping rates were lower than alluvial well pumping rates. The pumping rates were thought to range from 0 to 10 gpm (Prusinski 1988). However, pumping durations were not known.

The Denver wells were installed and pumped after start-up of the full-barrier system. Pumping occurred in 1982 and was reported to last about two years. The Denver wells were pumped to remove contamination that was detected in the Denver Formation. Pumping ceased in 1984 after it was feared that this pumping was possibly inducing additional contaminant migration into the Denver Formation.

Since reliable data on Denver well pumping rates was not available, a range in pumping rates of from 0 to 20 gpm was used in the model. This was distributed uniformly among all of the Denver wells.

5.3 Model Simulations

As stated before, the scarcity of data limited the objectives of this 3-D modeling effort to theorize on potential contaminant migration pathways and evaluating the feasibility of a more detailed 3-D modeling effort. The predominant questions prompting this part of the study were: (1)How did contamination reach the Denver sands units in the vicinity of the North Boundary Barrier System?; and,(2) Where did it originate? The approach used was to evaluate the relative likelihood of contaminants reaching the Denver

Formation in the vicinity of the North Barrier System through vertical leakage from the alluvial aquifer or from lateral migration from contaminated areas south of the Barrier System. Thus, two different types of simulations were performed. The first type was performed to investigate vertical leakage from the alluvial aquifer and the breakthrough of contaminants through the weathered clay shale. The second type investigated lateral migration of contaminants from upgradient sources (i.e. Basin F area). The question to be answered here is was more likely responsible for the groundwater contamination observed in the Denver Formation in the vicinity of the North Barrier System.

Diisopropylmethylphosphonate (DIMP) was selected as the contaminant to be modeled. DIMP occurs in higher concentrations than any other contaminant in the alluvial or Denver Sands Unit. As in other studies (Robson and Warner , 1977; Kessler, 1982), DIMP was modeled as a conservative substance, i.e. adsorbtion and other attenuation mechanisms were ignored. Based on 1984 to 1988 DIMP concentrations maps, DIMP has a fairly uniform concentration of 1000 mg/l in the alluvial aquifer within the model study area. In the Denver Sand Unit, DIMP concentrations vary from 0 to 300 mg/l within the model study area. In all simulations the alluvial aquifer was modeled with a uniform concentration of 1000 mg/l for DIMP while the concentrations in the Denver Sand unit varied. Each simulation is discussed below.

5.3.a Model Simulations of Lateral Migration

These simulations were performed to investigate lateral migration of DIMP within the upper Denver Sand unit from the Basin F area (located at the South boundary of the model area). Simulations were made (5.3.a.1 to

5.3.a.7) with the results given in table 5.1. In these simulations, a slug of DIMP was introduced at the South boundary of the model area and allowed to migrate northward. Simulation 5.3.a.1 was run for average aquifer characteristics (see Section 5.2 for discussion).

Model calculations were that DIMP would not reach the North barrier system from the Basin F area through lateral migration through the Denver Formation for at least 160 years. This time period is much longer than the time period since waste practices at Basin F began (30 years) indicating that it is unlikely that lateral migration in the Denver Sand unit is the source for contamination existing near the North barrier system.

Simulations 5.3.a.2 and 5.3.a.3 were performed to determine the sensitivity of the model predictions to Denver Sand porosity. Simulation 5.3.a.2 was run with a Denver Sand Unit porosity of 0.20 representing the extreme lower range of porosity for this material. The model results are that contaminants would reach the North barrier system after a period of 112 years. This period of time is still longer than the period of time time that Basin F has been in use. For the upper range of porosity (0.40) simulation 5.3.a.3 indicates that contaminants reach the North barrier system after 198 years.

Simulation 5.3.a.4 was performed to determine the effect on model predictions if the sand unit transmissivity estimated from field tests was underestimated by an order of magnitude (ie. 43.0 ft^2/day instead of 4.3 ft^2/day). The model calculated a migration time of 33 years. This time period is near the range of time that waste disposal activities began.

Table 5.1
Lateral Migration Simulations

Parameters

Simulation No.	Comment	Sand Unit Transmmissivity ft^2/day	Porosity	Cumulative Denver Well Pumping Rate (gpm)	Time of Travel Years
5.3.a.1		4.3	0.33	0	160
5.3.a.2		4.3	0.20*	0	112
5.3.a.3		4.3	0.40*	0	198
5.3.a.4		43.0*	0.33	0	33
5.3.a.5*	Confining Layer Thickness Changed to 1 foot upgradient	4.3	0.33	0	132
5.3.a.6		4.3	0.33	10	25
5.3.a.7		4.3	0.33	20	18

* Not an average or conservative aquifer characteristic

However, it is unlikely that the transmissivity has been underestimated by an order of magnitude since it was derived from field tests on the Denver Sands.

Simulation 5.3.a.5 was performed to determine if it might be possible for contaminants to have migrated laterally through the alluvial aquifer for some distance then at some intermediate area to have migrated vertically through the clay-shale layer into the Denver formation and then to migrate laterally through the Denver formation. In this simulation the thickness of the confining layer was reduced to one foot over a large area located midway between Basin F and the North Barrier system. The model calculated that even under this scenario, it would still take 132 years for the contaminants to reach the North Barrier system.

Finally, two simulations were performed to determine the effect of Denver well pumping (simulations 5.3.a.6 and 5.3.a.7). In the first simulation the total Denver well pumping rate was 10 gpm. The wells were pumped at this rate (approximately 1 gpm for each Denver well) for a period of two years and then shut off. In this scenario, the model calculated that contaminants would reach the Barrier in 25 years. When the pumping rate is 20 gpm, the model calculated that lateral migration of contaminants could reach the barrier in 18 years. Since the Denver wells were pumped only during 1982-84 time period, not enough time has elapsed for this to be a feasible migration path for the existing groundwater contamination observed in the Denver wells in the vicinity of the North barrier system.

5.3.b Model Simulations of Vertical Migration of Contaminants Through the Weathered Clay Shale

These simulations were performed to investigate vertical contamination migration through the weathered clay shale separating the alluvial aquifer and the top Denver Sands Unit (NBW#14). The vertical hydraulic conductivity of the clay-shale layer was determined from a single pump test performed in the vicinity of the original pilot part of the North barrier system. Concern has been expressed over the adequacy of this test due its short duration. Furthermore, extensive fracturing and weathering has been reported to exist in this area. Assuming that the fracture existence is not uniform, variability in vertical hydraulic conductivity values in different areas may exist. Consequently, many of the simulations were performed to determine the sensitivity of model predictions to variations in the vertical hydraulic conductivity. Ultimately, it is desired to characterize migration times within a range of vertical hydraulic conductivity values.

In all of the following simulations, a uniform concentration value of 1000 mg/l was used for the alluvial aquifer while the bottom aquifer had an initial concentration of zero. The model was run until contaminants reached the bottom layer (breakthrough). The results of all breakthrough simulations are listed in Table 5.2.

Simulations 5.3.b.1 through 5.3.b.5 were performed to determine the range in breakthrough times that would occur for varying thicknesses of the clay silt layer and varying hydraulic conductivity and in the absence of man-induced stress (ie. pumping). In simulations 5.3.b.1 to 5.3.b.3, the model calculated breakthrough times a range in confining layer thickness from 4 to 16 ft using a representative vertical conductivity value. Table 5.2 indicates that natural breakthrough may occur between 4.4 and 12 years (depending on the thickness). In general, the model predicted that

Table 5.2
Breakthrough Simulations
Parameters

Simulation No.	Confining Layer Vertical Conductivity ft/day	Confining Layer Thickness (ft)	Cumulative Denver Well Pumping Rate (gpm)	Time of Travel Years
5.3.b.1	4.104×10^{-5}	8.0	0	9.4
5.3.b.2	4.104×10^{-5}	4.0*	0	4.4
5.3.6.3	4.104×10^{-5}	16.0*	0	12.0
5.3.b.4	4.104×10^{-4}* (in vicinity of the barrer)	8.0	0	3.4
5.3.b.5	4.104×10^{-3}* (in vicinity of the barrer)	8.0	0	1.2
5.3.b.6	4.104×10^{-5}	8.0	10	2.7
5.3.b.7	4.104×10^{-5}	4.0*	10	1.1
5.3.b.8	4.104×10^{-5}	16.0*	10	5.4
5.3.b.9	4.104×10^{-4}* (in vicinity of the barrer)	8.0	10	0.7
5.3.b.10	4.104×10^{-3}* (in vicinity of the barrer)	8.0	10	0.2
5.3.b.11	4.104×10^{-5}	8.0	20	1.7
5.3.b.12	4.104×10^{-4}* (in vicinity of the barrer)	8.0	20	.5
5.3.b.13	4.104×10^{-5}	8.0	10 (for 6 months)	2.9
5.3.b.14	4.104×10^{-5}	8.0	20 (for 6 months)	2.7
5.3.b.15	4.104×10^{-5}	8.0	0	11.4**
5.3.b.16	4.104×10^{-5}	8.0	10	3.5**
5.3.b.17	4.104×10^{-3}*	8.0	0	1.9**

* Not an average characteristic
** Time period for bottom aquifer to reach 100 mg/l

breakthrough would occur in the vicinity of the Barrier system where the head differences between the alluvial aquifer and the Denver sand unit are the greatest.

Simulations 5.3.b.4 and 5.3.b.5 were performed to predict the effect that underestimating vertical hydraulic conductivity of the clay-shale layer would have on breakthrough times. Simulation 5.3.b.4 was performed with a vertical conductivity value of one order of magnitude greater than the value calculated from the pump test. This value was only used in an area from the slurry wall to 500 feet upgradient. The thought her is that fracturing and weathering effects are greatest in this area and that the vertical hydraulic conductivity may be underestimated in this area. The model predicted that breakthrough would occur in less than half the time than it would for base conditions (simulation 5.3.b.1). Simulation 5.3.b.5 was run with a vertical conductivity value of 2 orders of magnitude greater than field estimations. This simulation was performed to indicate the effect of a gross-under estimation of vertical hydraulic conductivity. This simulation is also representative of a case where vertical conduits such as deep fractures or poorly constructed wells may exist (i.e. groundwater contamination caused by leakage down along the well casing). The model calculated that breakthrough would occur very quickly (1-2 years).

Simulations 5.3.b.6 through 5.3.b.10 were performed to model the effect of Denver well pumping on vertical contaminatant migration. In these simulations, wells were pumped uniformly at a total pumping rate of 10 gpm for a period of two years and then shut off to simulate historical pumping. Simulations 5.3.b.6 through 5.3.b.10 were similar to simulations 5.3.b.1 through 5.3.b.5 except that the Denver wells were pumped at a cumulative rate of 10 gpm (1.0 gpm each). The model calculated that breakthrough would

occur in the frame of .2 to 5.4 years. This indicates that pumping of the Denver wells could have had a significant effect on contamination migration from the alluvial aquifer. Even shorter breakthrough times were calculated when the Denver Sand Unit was pumped at a cumulative rate of 20 gpm (simulations 5.3.b.11 and 5.3.b.12).

Simulations 5.3.b.9 and 5.3.b.12 modeled the effect of increasing the vertical conductivity by an order of magnitude in the vicinity of the barrier system. Similarly, simulation 5.3.b.10 modeled the effect of a gross-underestimation of vertical conductivity (two orders of magnitude). All three of these simulations resulted in breakthrough times of less than one year. This seemingly indicates that where the vertical hydraulic conductivity may have been underestimated or deep fractures and weathering may be present, pumping the Denver wells could have induced contaminant migration in a relatively short period of time.

Arsenal personnel were somewhat uncertain about the period of time in which pumping of the Denver wells took place. A conservative pumping period of two years was used in most of the simulations in this study. However, simulations 5.3.b.13 and 5.3.b.14 were performed to simulate the effect of a shorter pumping period (i.e. 6 months). The results in Table 5.2 indicate that even if the Denver wells had been pumped for a relative short period of time, this was sufficient to induce vertical leakage through the clay silt layer in the order of three years.

In the previous simulation, only time of initial breakthrough of vertical migration through the weathered clay-shale layer was considered. Simulations 5.3.b.15, 5.3.b.16 and 5.3.b.17 were performed to determine the amount of time required for concentrations in the bottom aquifer to build to at least 100 mg/l. This value is in the range of concentration values

currently detected at the Arsenal. Simulation 5.3.b.15 was performed under natural gradients (ie. no pumping). In comparison with simulations 5.3.b.1, the model predicts that it would take about 11.4 years or about two years beyond breakthrough for concentrations to build to near 100 mg/l. If pumping occured at 10 gpm (simulation (5.3.b.16), it would also take only about 3.5 years to build to 100 mg/l. Finally, simulation 5.3.b.17 indicates that if vertical conductivity was much greater than the estimated value, it would only take 2 years for the concentration to build to 100 mg/l under natural gradients.

5.4 3-D Modeling Conclusions

This part of the study has involved the application of the quasi three-dimensional version of of the contaminant transport CSU-GWTRAN to the North Boundary Barrier System. A part of the 2-D model study area was used in this study to investigate vertical and lateral migration of DIMP to the vicinity of the original pilot scale system of the North barrier system. The study was performed to help evaluate possible explanations for the existence of contamination in the Denver Sands Units. Model predictions were made with very limited data for the Denver Sand Unit. Consequently, the results of this study should be used at most to understand the relative importance of the different migration pathways.

Model calculations indicate that lateral migration of contaminants from the Basin F area is very slow. In most cases, it would take contaminants much longer to migrate to the barrier system than the time since Basin F has been active. The model results indicate that it is more likely that the contamination existing in the Denver Formation in the vicinity of the original pilot scale system was the result of vertical leakage from the

alluvial aquifer in the nearby vicinity of the barrier system. Vertical migration times on the average, are much less than lateral migration times. If vertical conductivity in the vicinity of the Barrier System is increased to a value representative of fractured and weathered clay shale, breakthrough occurs much sooner. The model predicts that pumping could have induced vertical migration through the weathered clay shale. However, it is difficult to make conclusive remarks about pumping without better data.

The results discussed here should be viewed as relative. As shown in the 2-D study, more conclusive results are rendered from a well calibrated larger scale model. It is recommended that a larger scale model be constructed, calibrated and applied to the area in the vicinity of the North Boundary Barrier System to further investigate the contaminant migration pathways to the Denver Sands Units. Additional data would be needed as part of any 3-Dimensional modeling effort of the Alluvial aquifer and the Denver Sand Unit.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

CSU-GWFLOW has been successfully applied to modeling a very complex and dynamic groundwater flow system. The system was calibrated to groundwater conditions observed for the north boundary prior to installation of containment treatment system. The model was verified and further calibrated by simulating the actual operation of the barrier system through nine and one quarter years of record to the present day. The predictive capability of the model was tested and refined. The model was recalibrated to incorporate additional operation data and monitoring well data for 1987. The calibrated and verified model was utilized to investigate the existing capability of the system and to estimate the consequences of reconfiguration of the discharge and recharge well arrays and other augmentation schemes. It was also utilized to predict water table conditions resulting from system breakdown scenarios.

The ultimate goal of management personnel at the Arsenal is to maintain a reverse gradient across the entire length of the slurry wall for the North Boundary Barrier System; a reverse gradient being defined as a hydraulic gradient in the opposite direction to the natural pre-barrier northward flow. This modeling study has shown that the current contraints to the attainment of this goal at the North Boundary include:

- Existing discharge and recharge well spacing in relation to each other and to the slurry wall barrier.

- Natural (pre-barrier) flow patterns in relation to placement of the barrier system.

- Constrained recharge capacity due to natural geohydrologic factors.

Although not modeled, observed recharge well inefficiencies due to clogging of carbon fines, may also hamper the effort to attain a reverse gradient.

In short, a reverse gradient along the entire length of the slurry wall is not possible with the present well configurations and recharge rates. A reverse gradient could be approached if the system is overpumped for a prolonged period of time. However, this is not considered feasible due to the fact overpumping eventually depletes the aquifer making it difficult to maintain large pumping rates.

Historical discharge and recharge rates have been estimated by the model. For comparison, natural flow rates to and from the barrier have also been estimated. This study has shown that the historical operation of the barrier system is not consistent with natural flow conditions. Many wells have been overpumped while others have been underpumped, resulting in undesirable gradients across the slurry wall. The gradient is highest over manifold A which intercepts most of the highly contaminated groundwater. This result is more a reflection of historical recharge than discharge. The fact that the recharge wells are all connected to one common manifold makes it difficult to regulate specific recharge areas. Consequently, areas such as the bog are preferentially recharged. The manifold C overpumping scheme is also a cause for the high gradients on the westerly portions of the barrier.

This study has shown that it is more desirable to operate the system at natural flux rates. This scheme results in a lower gradient over the most effected region of the barrier and a more equitable distribution of head differential consistent with pre-system conditions. Table 6.1 lists the historical and natural discharge and recharge rates and highlights the areas that deviate from natural interception. The natural rates listed in Table 6.1, should not be viewed as exact operation criteria; they should be viewed more as relative values for the corresponding region of the barrier that should be approached.

Operation of the system at natural flow rates is more desirable; however, it still does not result in a reverse gradient. It is apparent that in order to approach a reverse gradient over all or part of the slurry wall, a major well reconfiguration or augmentation scheme would be required. This study has examined several schemes. The most favorable scheme appears to be one in which both arrays of wells are moved closer to the barrier and operated at natural flux rates. The model estimates that the gradients are decreased an average of 3.5 feet over the entire length of the slurry wall under this scheme.

It should be noted that although the model results indicate that a reverse gradient can be nearly attained under this scheme, in practice, the occurrence of clogging and the inability to accurately regulate individual recharge and discharge rates may hamper this effort.

Augmentation of the system by recharge trenches has been modeled in this study. Such a scheme may prove to be most desirable since it might be difficult or impossible to attain and maintain a reverse gradient over the entire length of the slurry wall. It may be more feasible to concentrate

Table 6.1

Natural vs. Historical Recharge and Discharge Rates

Discharge Wells				Recharge Wells			
Discharge Well	Natural (gpm)	Historical (gpm)	Comment	Recharge Well	Natural (gpm)	Historical (gpm)	Comment
Manifold A							
330	2	2.0		432	0.0	1.5	OR
331	2	0.5	UP	433	1.0	1.0	
332	1	1.5	OP	434	0.5	0	UR
333	1	0.5	UP	435	0	0	
334	1	0	UP	436	0.5	1.0	
335	1	0	UP	437	1.0	0.5	UR
301	2	1.0	UP	438	1.5	0.5	UR
302	5	1.5	UP	401	0	0	
303	7	0	UP	402	0.5	0.5	
304	10	11.5	OP	403	2.5	1.0	UR
305	13	8.0	OP	404	3.0	1.5	UR
306	10	12.5	OP	405	5.0	1.0	UR
				406	3.5	0.5	UR
				407	7.0	0.5	UR
Manifold B				408	1.0	2.5	OR
307	6	0	UP	409	7.5	2.5	UR
308	6	0	UP	410	7.0	4.5	UR
309	6	0	UP	411	6.0	3.0	OR
310	7	0	UP	412	1.0	1.0	
311	5	0	UP	413	5.0	2.5	UR
312	5	0	UP	414	3.0	1.5	UR
313	8	16.0	OP	415	8.0	1.5	UR
314	8	17.0	OP	416	6.5	6.5	
315	3	7.0	OP	417	5.0		
316	4	10.0	OP	418	15.0		
317	5	7.0	OP	419	8.0	105	OR
318	7	13.5	OP	420	5.5		
Manifold C				421	22.0		
319	13	8.5	UP	422	16.0	9.0	UP
320	12	0	UP	423	0	8.0	OR
321	17	20.0	OP	424	25.5	6.5	OR
322	12	21.0	OP	425	13.0	15.5	OR
323	16	26.0	OP	426	2.0	8.0	OR
324	9	5.5	UP	427	5.0	15.0	OR
325	6	12.0	OP	428	19.0	3.5	UR
326	3	11.5	OP	429	4.5	2.5	UR
327	2	3.0	OP	430	5.5	6.5	OR
328	4	0	UP	431	1.0	6.5	OR
329	1	0.5	UP				

Note: OP - Over-pumped UP - Under-pumped
OR - Over-recharged UR - Under-recharged

efforts on attaining a reverse gradient over the region of the barrier that intercepts the most highly contaminated water (i.e. manifold A). The results of the trench simulations indicate that operation of the trenches at over 100 gpm could result in a reverse gradient over all of manifold A. However, it is very conceivable that the clogging observed in recharge wells, could also be observed in recharge trenches and thus reduce recharge capabilities.

Operational breakdown scenarios have also been examined in this study. The model has estimated where and when water will overtop the slurry wall under various simulations. In the worst case, where the plant is flowing at approximately 220 gpm and instantaneously fails, it appears that overtopping will occur near First Creek about two weeks after plant shut down. Failure of only Manifold C wells results in a slightly longer overtopping time. Failure at a plant flow of 300 gpm (assuming flow to the barrier is approximately 220 gpm) results in a longer time before overtopping occurs (i.e. 34 days). During the late winter and early spring when the flow to the barrier might be higher, overtopping will occur sooner. In all cases, the model predicts that overtopping will only occur near First Creek. The model results indicate that this is the most vulnerable region of the barrier.

The Arsenal personnel were an important part of the modeling efforts. Their intimate knowledge of the site, monitoring well network, and north boundary system was invaluable in calibration, verification, and implementation of the model. It should be noted that like most, the science of groundwater modeling is inexact. The results of any modeling effort should be used at most, to gain an understanding of a very complicated problem. It is hoped that our efforts will head to a better understanding

of the complex hydrology of the RMA and improved operations of the north boundary barrier system. However, the decision to implement an alternative should be based on practical considerations, technical merits, the cost benefits, and evolving goals of the effort.

The three-dimensional modeling that was conducted as part of this study was useful in evaluating possible migration pathways by which the observed groundwater contamination in the Denver Formation in the vicinity of the North Barrier system may have resulted. In general, model simulations indicate that lateral migration of contaminants in the Denver Formation is not the likely source of this observed contamination. More likely it is the vertical migration of contaminants from the alluvial aquifer through the weathered clay-shale layer to the underlying Denver Formation. This vertical leakage most likely occurs in the nearby vicinity of the North Barrier system where the clay-shale layer may be more fractured or weathered than elsewhere. Natural vertical migration may have resulted in the initial groundwater contamination observed in the Denver Formation. This most likely prompted Arsenal personnel to pump the Denver wells to try to remove this contamination. The extent to which this pumping induced additional vertical migration of contaminants is uncertain. Equally uncertain is the role that poor well construction may have had in providing an additional migratory pathway for the contaminants to have entered the Denver Formation.

6.2 Recommendations

Results of numerous model simulations have lead to the following suggestions and recommendations:

1. Installation of accurate recording flow meters which would allow the Arsenal to review the operation of individual wells in the system. knowledge of individual well operations is critical for implementation of individual pumping rates which are essential in attaining the Arsenal's management goals. Installation of continuous recording flow metering would allow the Arsenal to document changes in barrier operation for periods of time when an operator is not present.

2. Resumption of regular water level surveys for the model area, to allow for anticipation of changes in flow to the north boundary due to planned and on-going remedial activities at the RMA. Re-establishment of well reference elevations should be performed to account for wells that have been fire-damaged or otherwise disturbed since installation. Individual water level surveys should be performed over a short period of time to obtain an instantaneous picture of potentiometric conditions in the surficial aquifer. Water level measurements for wells immediately adjacent to the barrier system should be accompanied by system operations information to document cause and effect of recent system operations.

3. Observation and analysis of data from the monitoring wells installed in pairs across the slurry wall to document head differentials across the system. Use of information from monitoring wells at some distance up, and down gradient of the slurry wall gives an incorrect picture of the head differential across the barrier. This information will be useful in field verification of model predictions. Before and after effects of barrier reconfiguration schemes could be documented. Installation of continuous water level recorders for these wells would allow documentation of slurry wall head differentials for RMA and regulatory use.

4. Because the distribution of contaminants across the barrier alignment is important to operation of the individual manifolds and the system as a whole, contaminant transport simulation should be performed to examine the effects of system operation on the distribution of contaminants intercepted by the barrier. Contaminant transport modeling would allow review of the long term implications of the current manifold C overpumping scheme being employed at the north boundary. Effects of remedial activities such as abandonment of basin F and near-source interception systems could be reviewed. Dissipation of off-post residual contaminant concentrations over time could be examined.

5. Perform an as-built review of the pumping, recharge, and treatment system piping to evaluate the effects of conveyance efficiency on performance of pumping and recharge wells. This field effort would be best performed after installation of the individual well recording flow meters as outlined in recommendation number 1 and could include:

As-built survey of conveyance system invert elevations and pressure testing of the collection and delivery laterals under typical operating conditions to evaluate the distribution of hydraulic head across the system. Comparison of hydraulic head information with typical observed and model computed equilibrium flow rates of individual wells may give an indication of the effects of distribution system design on barrier operation.

Time-operations study of the system to review the effects of cycling of system components on overall pumping and recharge performance. This study could include installation of continuous water level recorders to examine fluctuations of local potentiometric levels due to system cycling.

6. If the Arsenal elects to pursue reconfiguration of well arrays or augmentation by trenches to improve head differentials across the slurry

wall, it is recommended that they consider a pilot program of test drilling to review the extent of migration of the bentonitic materials into the aquifer and discharge/recharge capacity of wells or trenches located closer to the slurry wall. This program could proceed on a manifold by manifold sequencing to allow continued operation of the system during construction.

For any reconfiguration scheme selected, the Arsenal should consider segmentation of the recharge manifold into sections with separate effluent pumps from the common effluent wetwell. This would result in more complexity of system operation, but would allow more flexibility of operation of recharge component. Recharge in-kind across the slurry wall from pumping manifolds would likely be more attainable under this configuration. The ability to provide maintenance down-time for segments of the recharge wells would allow re-development cleaning of recharge wells on a manifold by manifold basis. Breakdown of one section of recharge manifold would not require complete system shutdown.

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